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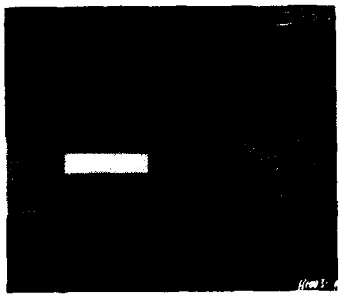

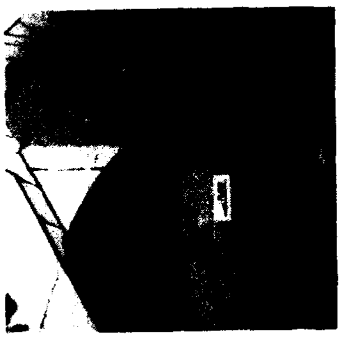

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-92-15

CENTER HILL FUSEPLUG SPILLWAY CANEY FORK RIVER, TENNESSEE

Hydraulic Model Investigation

by

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Model tests of the Center Hill fuseplug spillway mounted on a fixed crest at Caney Fork River, TN, were conducted to develop a satisfactory design that would fail in a orderly fashion at a prescribed rate. Different zones in the prototype fuseplug (such as the upstream and downstream slope protection, the impervious core, and the filter) were represented in the model by different soil types. Model materials were processed and manufactured to simulate certain characteristics of the prototype soil based on sound modeling theory, so that behavior in the model would simulate behavior in the prototype. Grain-size distributions of the granular materials were used to select appropriate grain size of the corresponding model material. Determination of the model grain sizes was made by matching the terminal particle settling velocity (through water) between model and prototype based on Froude numbers. The impervious core in the model consisted of prototype clay taken from the embankment site. Three physical models were used to screen and investigate various

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designs and to determine if scale effects would significantly influence hydraulic performance of the models. Test results indicated that scale effects were insignificant. A fuseplug design was developed in which the 600-ft-long spillway would fail at a constant rate in approximately 29 min. The models also permitted determination of discharge coefficients for the fixed crest and the magnitude and direction of currents in the exit area.

PREFACE

The model investigation reported herein was authorized by the Headquarters, US Army Corps of Engineers (HQUSACE), on 19 December 1990 at the request of the US Army Engineer District, Nashville (ORN). The studies were conducted by personnel of the Hydraulics (HL) and Geotechnical (GL) Laboratories of the US Army Engineer Waterways Experiment Station (WES) during the period December 1990 to February 1992 under the direction of Mr. F. A. Herrmann, Jr., Director, HL; Dr. William F. Marcuson III, Director, GL; R. A. Sager, Assistant Director, HL; and Dr. Paul F. Hadala, Assistant Director, GL; and under the general supervision of Mr. G. A. Pickering, Chief, Hydraulics Structures Division (HSD), HL; Dr. Don Banks, Chief, Soil and Rock Mechanics Division (GS), GL; Mr. N. R. Oswalt, Chief, Spillways and Channels Branch, HSD; and Mr. G. P. Hale, Chief, Soils Research Facility, GS. Project Engineers for the model studies were Messrs. B. P. Fletcher, HSD, and P. L. Gilbert, GS. Technicians assisting the study were Messrs. R. B. Bryant, J. R. Rucker, and E. L. Jefferson, HSD; and C. E. Carter, GS. This report was prepared by Messrs. Fletcher and Gilbert and edited by Mrs. M. C. Gay, Information Technology Laboratory, WES.

During the investigation, Messrs. Sam Powell, HQUSACE; Lyn Richardson, Dave Hammer, and Russ Fondelier, US Army Engineer Division, Ohio River; Jim Paris, Gary House, Ben Couch, John Hunter, Hank Phillips, Wayne Huddleston, Paul Bluhm, and Gordon McClellan, ORN; and Cliff Pugh, US Bureau of Reclamation, visited WES to discuss the program of model tests and observe the models in operation.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
inches	25.4	millimetres
inches of mercury (32 °F)	3.38638	kilopascals
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

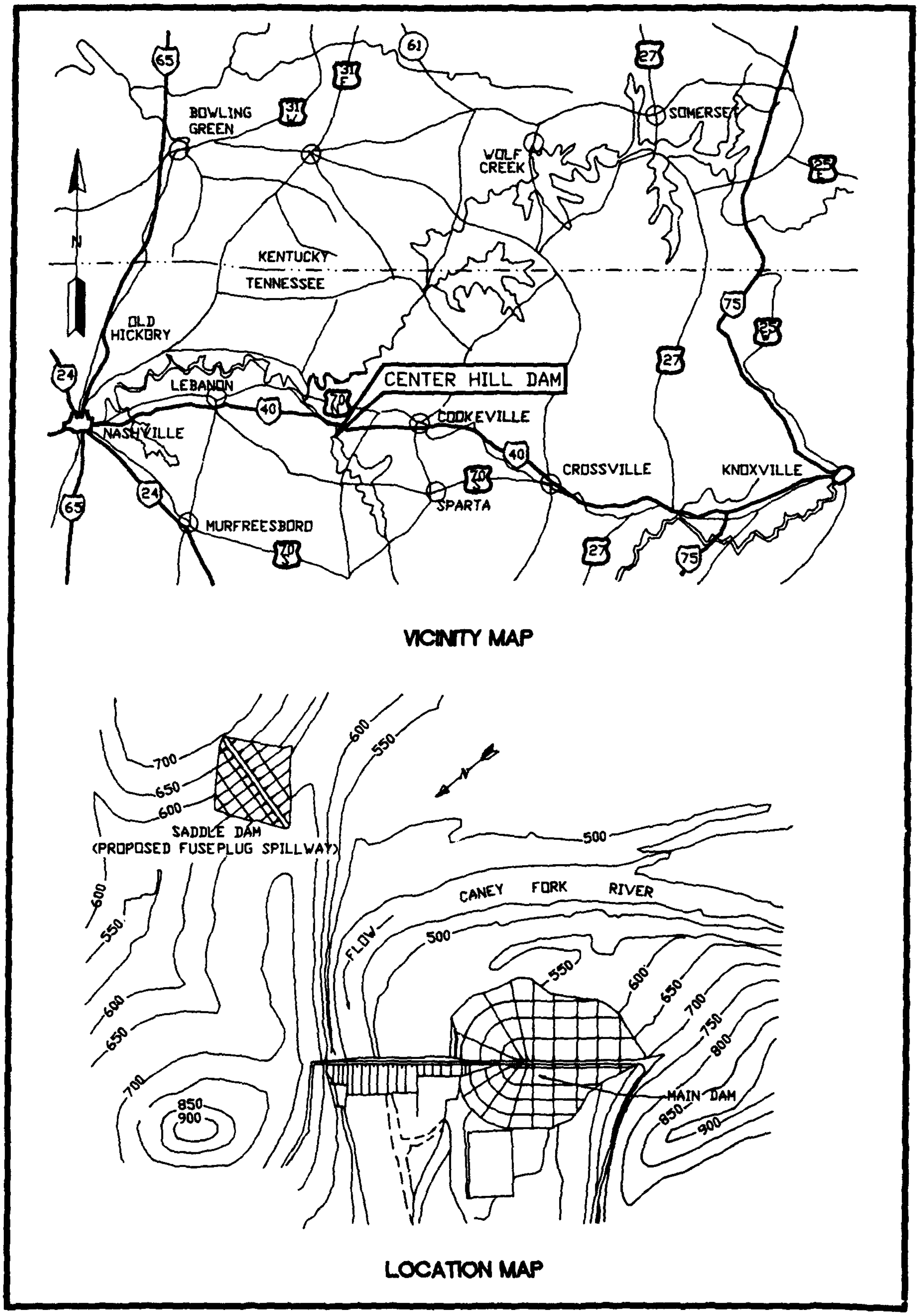

Figure 1. Vicinity and location maps

CENTER HILL FUSEPLUG SPILLWAY

CANEY FORK RIVER, TENNESSEE

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. A fuseplug is a zoned earth structure designed to wash away under specific hydrologic conditions in a controlled manner to provide additional spillway discharge when the capacity of the existing service spillway and outlet works associated with a permanent embankment is exceeded. Fuseplugs are usually designed and constructed to resist/withstand indefinite cycles of reservoir water level rising up to the crest; however, if the crest is overtopped, the structures are intended to be quickly removed.

2. Because of topographical constraints around the Center Hill Dam in northern middle Tennessee, the US Army Engineer District, Nashville, determined that a fuseplug structure is a reasonable and feasible means of providing emergency spillway capacity. Since the location and timing of a fuseplug washout must be controlled and predictable, it was decided to conduct physical model tests to investigate design, performance, and expected behavior characteristics in the fuseplug structure to be placed at Center Hill Dam. The model study of the Center Hill fuseplug spillway was a joint venture involving the Hydraulics and Geotechnical Laboratories of the US Army Engineer Waterways Experiment Station (WES).

Prototype

3. The Center Hill Dam is located on the Caney Fork River, 26.6 miles* above its confluence with the Cumberland River. The site is located in Tennessee, 55 miles east of Nashville and 19 miles southwest of Cookeville (Figure 1).

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* A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

4. The existing dam is 2,160 ft long, which includes a concrete gravity dam with a crest length of 1,382 ft and a 778-ft-long rolled filled embankment with a top elevation of 696.0.* The concrete dam consists of a 470-ft overflow spillway, a 245-ft abutment section on the right side, a 267-ft power intake section on the left side, and a 400-ft section tying into the embankment. The ogee spillway crest elevation is 648.0 and consists of eight 50-ft-wide by 37-ft-high tainter gates.

5. About 1,800 ft upstream from the dam on the right reservoir rim is a rolled fill dam (saddle dam) 775 ft long with a crest elevation of 696.0 (Figure 1). A study was made that revealed that the dam is not safe for floods exceeding 72 percent of the possible maximum flood (PMF). Several alternatives were developed that could safely pass the PMF. The most economical and recommended plan consisted of replacing the saddle dam with a fuseplug spillway (Plate 1).

6. The 34.4-ft-high fuseplug spillway was designed to be mounted on a concrete-lined fixed spillway with a crest elevation of 692.4 and invert length of 600 ft (Plate 1). The fuseplug spillway was designed to be a stable structure except when overtopped. When overtopped, it should erode from the surface of the fixed crest in 30 min leaving a concrete spillway with a 600-ft-long crest. A typical section (final design) through the fixed crest and fuseplug spillway is shown in Plate 1. A clay core 5.66 ft thick and inclined at a 45-deg angle to the downstream will provide an impervious barrier to el 692.4. To initiate erosion of the fuseplug, a pilot channel will be located in the center of the fuseplug spillway. The invert of the pilot channel will be 50 ft wide and 0.9 ft deep. Wave barriers will be installed to prevent premature failure by overtopping waves.

Purpose and Scope of the Model Studies

7. The model studies were conducted to investigate the best method of triggering initial erosion of the spillway, design of an impervious core constructed of clay and/or a geomembrane, size of pilot channel, size and location of wave barriers, optimal grain size of the shell material, vertical and

* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

lateral erosion rates of the spillway that will provide a 600-ft-long complete washout at the invert of the fuseplug spillway in 30 min, and flow characteristics and discharge coefficients of the fixed crest. Tests were conducted for anticipated flow and water-surface elevations. The design developed by the models should ensure that the structure performs satisfactorily.

PART II: THE MODELS

Description

8. Three physical models (Models A, B, and C) were used to investigate the hydraulic and geotechnical characteristics of the proposed fuseplug spillway. These three models complemented each other and permitted the evaluation of scale effects.

9. Model A was constructed to a 1:40 scale and simulated a 40-ft-long section of the fixed crest and fuseplug spillway in a 1-ft-wide, 2-ft-deep, and 30-ft-long flume (Plate 2 and Figure 2). The sides of the flume were transparent to permit observation of flow patterns, failure of the geomembrane and/or clay core, and vertical erosion of the granular material in the spillway. Model A was used as a two-dimensional screening tool for qualitative guidance in operation of the larger models (Models B and C) as to how failure by overtopping can best be initiated and how vertical erosion can be induced to proceed at a prescribed rate.

10. Model B was constructed to a 1:40 scale in a 12-ft-wide flume (Plate 3 and Figure 3). Model B simulated the right abutment (80-ft length), the pilot channel, a 400-ft length of the fixed crest and fuseplug spillways, and 800- and 600-ft lengths of the approach and exit channels, respectively (Plate 4). The approach and exit channel lengths were measured from the lateral center line of the fuseplug spillway. Model B was used to determine the vertical and lateral erosion rates of the most favorable designs developed in Model A. After complete washout of the fuseplug spillway, tests were conducted to determine the discharge and abutment coefficients and flow patterns for the fixed-crest spillway.

11. Model C was constructed at a 1:20 scale in the 12-ft-wide flume. The model simulated the center 240 ft of the fixed and fuseplug spillways (Figure 4 and Plate 5), the pilot channel, and 400- and 300-ft lengths of the approach and exit channels, respectively. The model simulated and permitted investigation of all features proposed in the investigation of Model B except for the hydraulic conditions generated by the right abutment. The primary purpose of Model C was to compare its erosion rate with that measured in Model B to determine if scale effects influenced the results obtained from Model B.

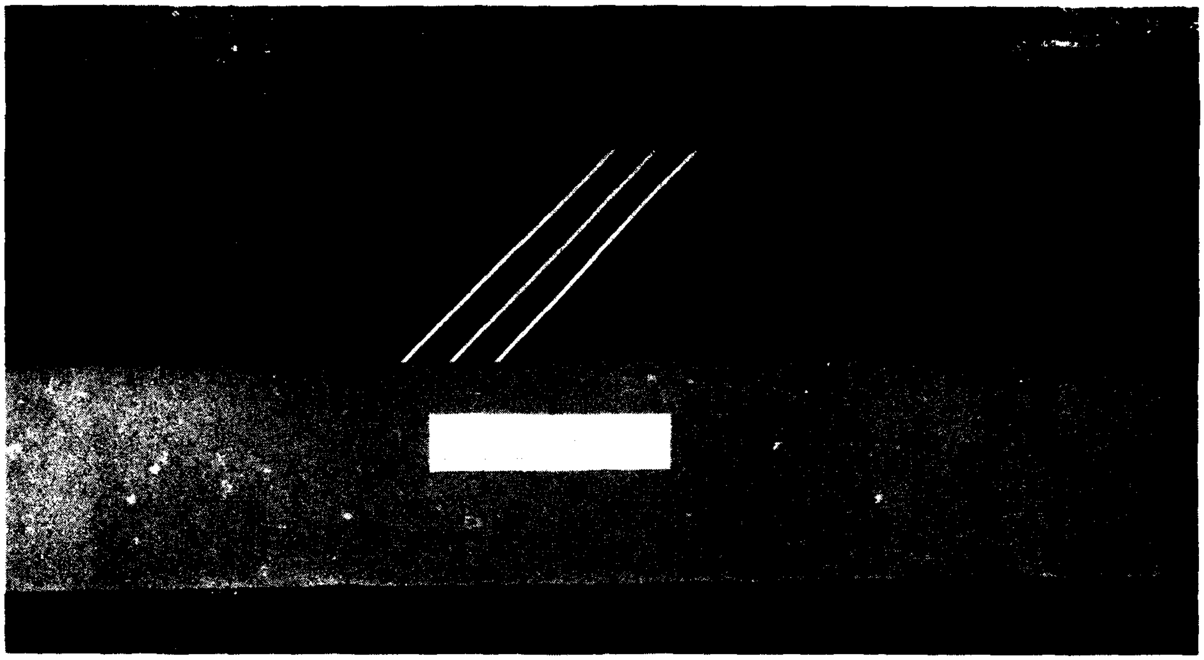

Figure 2. The 40-ft-wide section model (Model A)

Figure 3. The 480-ft-wide model (Model B)

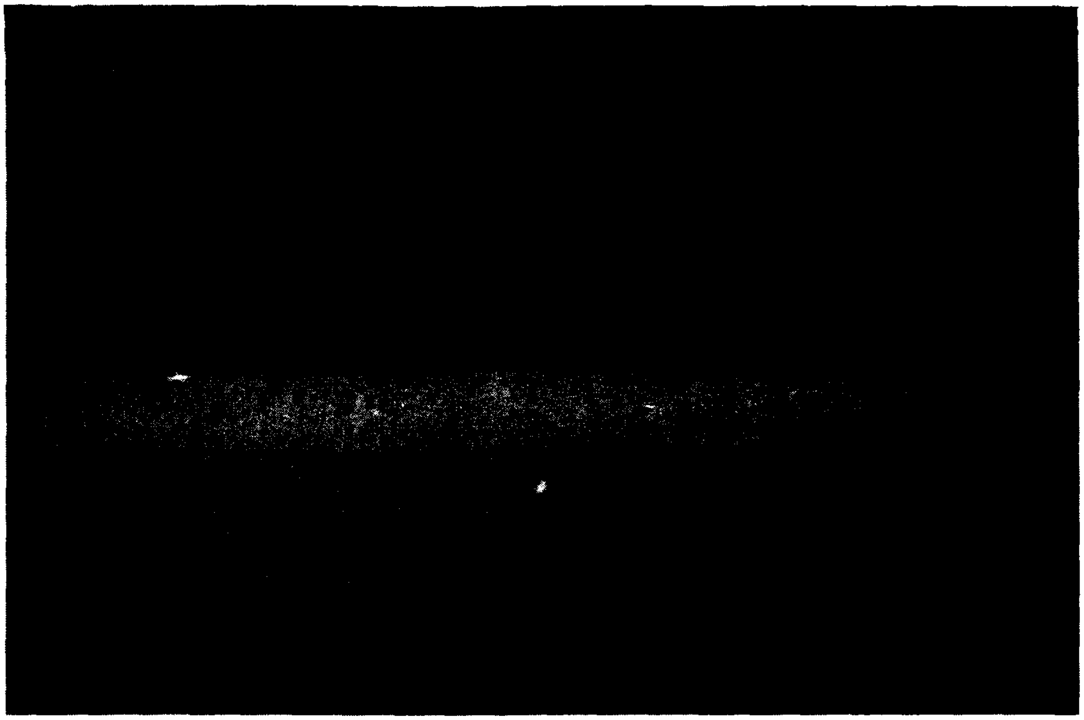

Figure 4. The 240-ft-wide model (Model C)

12. Water used in the models was recycled and discharges were measured with a weir and venturi flowmeters. Water-surface elevations were measured with staff and point gages and a pressure transducer. Current velocities were measured with pitot tubes. Fuseplug spillway erosion rates were measured with timing lines located on the spillway and a stopwatch. Videos of erosion enhanced the measurement of erosion rates. Flow conditions were documented by sketches, photographs, and videos.

Scale Relations

13. The accepted equations of hydraulic similitude based on Froude criteria were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and prototype. The general relations expressed in terms of the model scales or length ratios L_r are presented in the following tabulation:

Characteristic	Dimension*	Scale Relations Model:Prototype Models A and B	Model C
Length	$L_r = L_r$	1:40	1:20
Area	$A_r = L_r^2$	1:1,600	1:400
Time	$T_r = L_r^{1/2}$	1:6.3246	1:4.4721
Velocity	$V_r = L_r^{1/2}$	1:6.3246	1:4.4721
Discharge	$Q_r = L_r^{5/2}$	1:10,119.36	1:1,788.84
Weight	$W_r = L_r^3$	1:64,000	1:8,000

* Dimensions are given in terms of length.

14. Model measurements can be transferred quantitatively to prototype equivalents using the preceding scale relations.

PART III: GEOTECHNICAL LABORATORY TESTS AND RESULTS

15. All Geotechnical Laboratory tests were performed in accordance with the US Army Corps of Engineers Laboratory Soils Testing Engineer Manual (EM) 1110-2-1906.*

16. The following tests were performed on the sand and clay:

Sand	Clay
Specific gravity	Specific gravity
Maximum density	Atterberg limits
Minimum density	Combined grain-size and hydrometer analysis
Grain-size analysis	

A five-point compaction curve was developed for the clay using standard compactive effort (as defined in American Society for Testing and Materials (ASTM) Standard D 698**), which consists of compacting the soil in a 6-in.-diam mold with a 5.5-lb (sleeve) rammer falling 12 in. Specimens were compacted in three layers of equal thickness with 56 blows applied to each layer. A locally available (Vicksburg, MS) commercial crushed limestone gravel was obtained at the request of the US Army Engineer District, Nashville, to serve as the model upstream gravel and slope protection material for which the Nashville District furnished the required grain-size distribution curves.

17. Tests were performed by the Geotechnical Laboratory to characterize the soils of which the prototype fuseplug is to be constructed to produce appropriate materials for use in the small-scale models. Tests performed on the prototype materials consisted of the following:

- a. Sand fill: Relative density tests performed on the sand fill determined maximum density of 108.4 lb/cu ft and minimum density of 88.3 lb/cu ft. The Nashville District required that the sand fill be placed in the prototype at 70 percent relative density. In Froude modeling, density scales directly from model to prototype, so the sand fill was placed in the model at 70 percent

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* Headquarters, US Army Corps of Engineers. 1970 (30 Nov). "Laboratory Soils Testing," EM 1110-2-1906, US Government Printing Office, Washington, DC.

** American Society for Testing and Materials. 1991. "Standard Test Method for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop," Designation: D 698-78 (Reapproved 1990), Section 4, Construction, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics, Philadelphia, PA, pp 162-166.

relative density, which corresponded to 101.5 lb/cu ft. The grain-size distribution curve for the prototype sand as received from the Nashville District is shown in Plate 6. Specific gravity of the sand fill was 2.67.

b. Clay: A five-point compaction test on the clay received from Nashville showed the optimum density and water content to be 107.5 lb/cu ft and 18.0 percent, respectively; the compaction curve is shown in Plate 7. The grain-size distribution curve of this clay, given in Plate 8, shows that 5.5 percent of the soil is coarser than the No. 4 sieve. This plus No. 4 material was screened out of the soil and discarded before it was used for construction of small-scale models. The specific gravity Gs of the material is 2.72 and the liquid limits (LL) and plastic limits (PL) are 42 and 15 percent, respectively.

Froude Modeling

18. In Froude modeling, the dimensionless ratio of the square root of the inertia force acting to a control weight of material is matched between model and prototype. In equation form,

$$Fr = \frac{V}{\sqrt{gL}} \tag{1}$$

where

Fr = Froude number

V = water velocity

g = acceleration due to gravity

L = a characteristic length within the model

It must be stated that if micro dimensions within the prototype (e.g., the dimensions of soil particles) were scaled down exactly along with macro dimensions, then model soil particles would be so small that the dimensionless ratio of inertia force to viscous force (i.e., Reynolds numbers) would be severely distorted between model and prototype. In such a case, it can readily be seen that the resulting soil particles would be so small and drag forces so disproportionately large that particle settlement velocities in the model would be much too small. Therefore, certain adjustments to soil grain-size distribution must be made in classical Froude modeling to better match viscous force similitude between model and prototype. One way to achieve this similitude is to scale settlement velocities directly between model and prototype.

Model Material Determined by Settlement Velocity

19. The small-scale models in this testing program were constructed primarily of two granular soils: (a) a manufactured material consisting primarily of a fine fraction of commercially available Ottawa silica sand from Ottawa, IL, and a small fraction (approximately 10 percent) of a concrete sand, also commercially available in Vicksburg, MS; and (b) a commercially available limestone gravel whose grain-size distribution was specified by the Nashville District to serve as upstream gravel for the fuseplug structure.

20. The Ottawa sand used, called F-125 sand, was chosen because it is rich in the finer fractions needed to produce the model material; another very desirable feature of this sand is that its grains are rounded and visually appear to match the rounded grains of the prototype material. The prototype material is a quartz sand that occurs naturally at the embankment site in north-central Tennessee.

21. The following procedure was used to determine model grain-size distribution from the prototype material:

a. The grain-size distribution of the prototype material was determined.

b. Small samples of material from each sieve size used for the grain-size determination were taken and placed in individual glass containers with an amount of water sufficient to completely cover the soil.

c. High vacuum (about 29.5 in. of mercury) was applied to the soil/water mixture in the containers to remove all air from the soil.

d. A few soil particles from a given container (with a grain diameter determined by the sieve on which the particles were retained) were transferred to a transparent Plexiglas cylinder that was 8.5 in. in diameter and 48 in. long and was filled with water. The long dimension was aligned with the direction of gravity. The soil never came into contact with air during transfer to water within the cylinder (thus maintaining water saturation of the particles).

e. A few soil particles were released at a mark near the top of the water-filled cylinder and allowed to settle through 40 in. of water. The settlement time was measured with a stopwatch. From measured distance and time, settlement velocity was determined.

f. Settlement velocities were determined for the range of sieve sizes spanned by the material under examination and a log-log plot of settlement velocity versus particle diameter prepared. The relationship between settlement velocity and particle diameter for the downstream sand fill and upstream gravel used in this study are shown in Plates 9 and 10, respectively. It should be noted that the relationship between terminal velocity and particle diameter is well established and contained in the general relationship between drag coefficient versus Reynolds number for spheres settling through a general viscous fluid under the action of gravity as shown in Plate 11. However, if the particles under consideration depart from perfect sphericity, as most soil particles do, then the relationship of Plate 11 should not be used; instead, a velocity/diameter relationship must be established for each material of interest if this procedure is to be effectively used to achieve better similitude between model and prototype.

g. Using the relationship represented in Plates 9 and 10 a prototype particle diameter desired to be converted to a model particle diameter was selected and the velocity corresponding to that diameter determined. The relationship between model velocity and prototype velocity in Froude modeling is

$$\frac{V_m}{V_p} = \sqrt{\frac{L_m}{L_p}} \qquad (2)$$

where

V_p = velocity in the prototype configuration

V_m = corresponding velocity in the model configuration

L_p = characteristic length in the prototype configuration

L_m = corresponding length in the model configuration

Therefore, model velocity may be determined from a known prototype velocity and scaling ratio.

h. The relationship established in Plates 9 and/or 10 was entered with the model velocity; the diameter corresponding to that model velocity is the model diameter.

i. By repeating these operations, the entire prototype grain-size distribution was transformed to allow determination of the model grain-size distribution.

22. Plate 12 shows the sand fill grain-size distribution for the 1:40-scale model as well as the 1:20-scale model with the prototype material grain-size distribution included for reference. Plate 13 shows the upstream

limestone gravel grain-size distribution for the 1:40-scale model along with the gradation curve specified by the Nashville District. The materials used to construct small-scale models were manufactured by separating large quantities of granular material judged to be similar to the prototype material in grain shape, then blending those separate fractions together in the correct proportion to achieve the desired model grain-size distribution.

Influence of Water Temperature

23. The temperature of the water through which particles settle will influence the resulting settling velocity and therefore the particle size distribution determined by the procedure outlined in the previous section. Therefore, it is worthwhile to investigate the sensitivity of the resulting particle size distribution to variation in water temperature. An equation for terminal settling velocity V of spherical particles may be determined to be

$$V = \sqrt[3]{\frac{4}{3}\frac{\nu R g (G_s - G_f)}{C_d}} \tag{3}$$

where

ν = kinematic viscosity of water, ft^2/sec

R = Reynolds number (dimensionless)

g = acceleration due to gravity, ft/sec^2

G_s = specific gravity of the soil particles (dimensionless)

G_f = specific gravity of the fluid (dimensionless)

C_d = coefficient of drag of the particles (dimensionless)

24. Specific gravity and viscosity of water are the factors in Equation 3 that will change with temperature. By using the relationship shown in Plate 11 for spherical particles and Equation 3 along with tables giving the variation in specific gravity and kinematic viscosity for water, settling velocities will increase with increases in temperature as shown in the following tabulation:

Increase in Water Temperature, °F		Increase in Settling Velocity, percent
From	To	
50	60	4.79
60	70	4.53
70	80	4.24
80	90	3.88

Spherical particles settle through water faster than particles of any other shape, so they represent an upper bound in settlement velocity. Therefore, change in velocity due to change in temperature for particles of nonspherical shape will be less than that of spherical particles. A change in velocity of about 5 percent will represent a change in particle diameter of much less than 1 percent, so the effect of temperature variation may be neglected for practical purposes. However, for completeness, it should be mentioned that water temperature in the flume tests conducted in this investigation varied from 64.9 to 76.6 °F.

Cohesive Strength

25. A significant element in the zoned embankment being modeled in this investigation is the clay core, which serves as a impervious barrier to water seepage through the structure. Clay is characterized by cohesive strength; one of the weaknesses of Froude modeling is that cohesive strength does not model well. It may be shown from analysis of the governing equations that cohesive strength will be too large in the 1:40-scale model. In an attempt to achieve similitude of cohesive strength, the thickness of the clay core was investigated in the models at thicknesses equal to one-half and one-third its correctly scaled value. In this way, strength of the clay core was reduced in an attempt to satisfy similitude although it is acknowledged that similitude is not satisfied by this measure and the amount of departure from perfect similitude for cohesive strength is unknown.

PART IV: HYDRAULICS LABORATORY TESTS

26. A typical test for the two-dimensional Hydraulics Laboratory model (Model A) consisted of raising the water surface upstream of the spillway to el 692.4 to permit an initial 0.4-ft depth of flow over the crest of the pilot channel. The prototype will have no tailwater pool; thus no tailwater elevation was set on the downstream side of the spillway. As the spillway eroded, the upstream water surface was held constant at el 692.4 by increasing the model inflow to compensate for the gradually increasing flow through the vertically expanding breech. The water surface on all models was measured 300 ft upstream from the center line of the fuseplug spillway. Models B and C (three-dimensional models) were operated similar to Model A except after breaching, the lateral erosion rate was measured using timing lines located on the downstream slope of the spillway. Video cameras were used on all models to document performance from downstream and profile views.

Model A

27. Tests were conducted in a section model at a 1:40 scale to qualitatively evaluate the failure characteristics of various designs for the pilot channel. Essentially Model A was used as a two-dimensional screening tool for guidance in operation of the larger three-dimensional models (Models B and C) as to how failure by overtopping can best be initiated and how the rate of vertical erosion can be induced to proceed at a prescribed rate. The time required for total failure was measured for each design tested. Total failure was defined as complete removal of the fuseplug from the fixed crest. This parameter (failure time) was used as a qualitative index of comparison for the designs investigated in the 1-ft-wide flume. The recorded failure times in the 1-ft-wide flume are not indicative of the failure times for tests conducted in the 12-ft-wide flume.

28. A cross section of the initial fuseplug spillway design investigated (type 1) is shown in Plate 14. Failure was initiated by erosion of the downstream slope protection and sand fill. As the sand fill was hydraulically removed from behind the clay core, the clay core was subjected to an increasing differential load and failed, as shown in Plate 15. The time required for failure was 45 min (prototype) as shown in the following tabulation:

Type	Time min*
1	45
2	41
3	60
4	60
5	142
6	57
7	100
8	70
9	26
10	19

* Time is documented in prototype values.

29. After testing the type 1 design, it was apparent that the time for failure should and could be reduced by facilitating the initiation of erosion on the downstream slope. The size of the riprap slope protection was reduced (type 2 design), as shown in Plate 16. The prototype time required for failure was 41 min.

30. Following tests of the type 2 design, representatives from the Nashville District stated that the top of the impervious core should extend to el 692.0 (type 3 design, Plate 17). Raising the clay core to el 692.0 significantly reduced the permissible initial unit discharge over the top of the clay core and thereby increased the time required for failure to 60 min (paragraph 28). Stages of failure are shown in Photo 1.

31. In an attempt to increase the rate of initial failure, the top of the fuseplug was lowered to el 690.0 and was surmounted by a piggyback fuseplug with its top at el 692.0 (type 4 design, Plate 18). Also, a sheet-pile wave barrier, designed by the Nashville District to prevent premature failure by wave action, was installed. The wave barrier is shown in Plate 18 and Figure 5. Significant head loss (about 0.2 ft) across the wave barrier was observed during initial overtopping. This increased the time for initial erosion at the top of the fuseplug. Tests indicated that the piggyback feature, even without the wave barriers, would not significantly reduce failure time (paragraph 28). It was suggested by personnel at WES that a floating wave barrier, which would not create significant head loss, be considered for

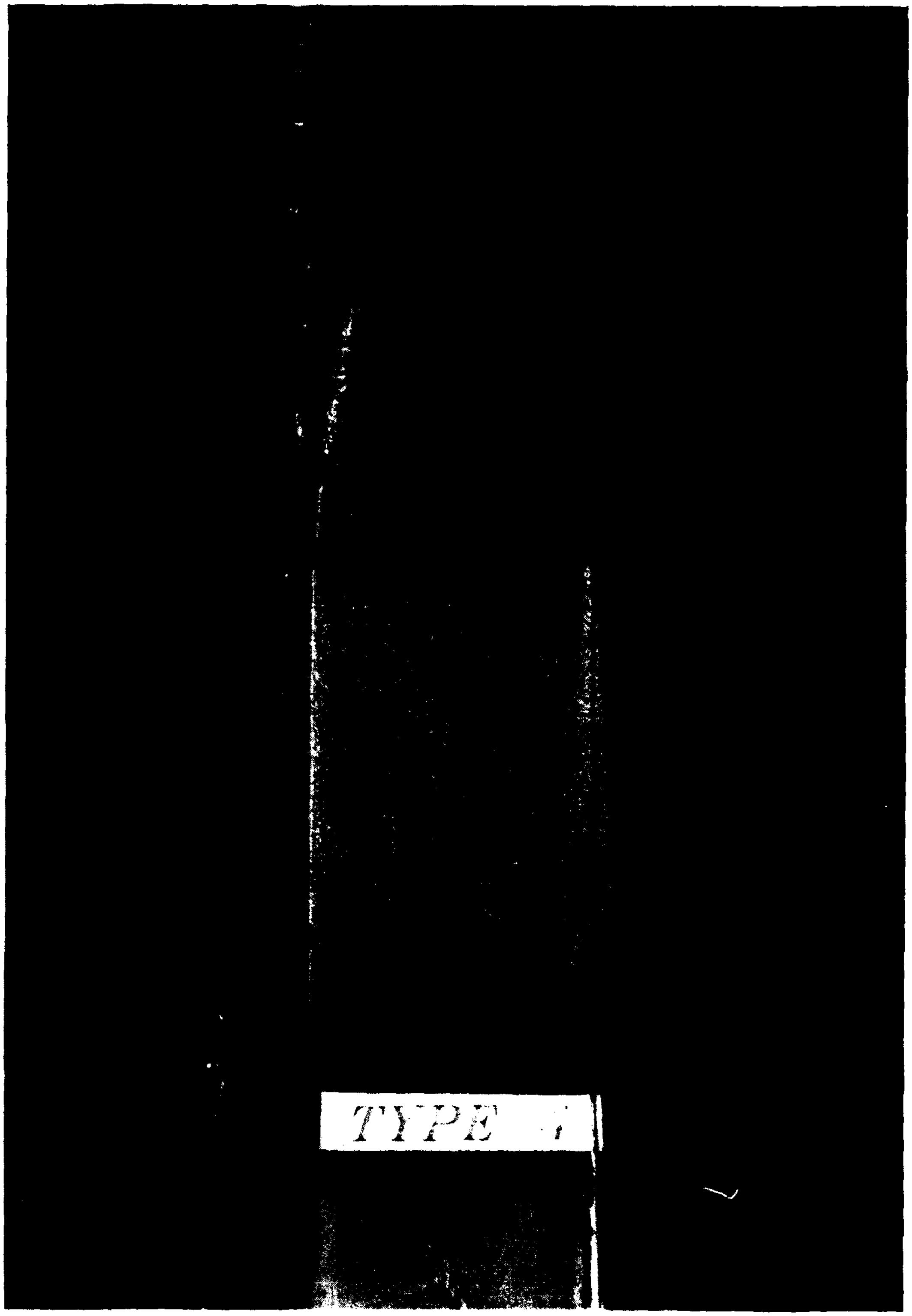

Figure 5. Sheet-pile wave barrier, Model A, type 4 design

the notched portion of the fuseplug and that further wave barrier tests be deferred until tests were conducted in Model B.

32. Initial overtopping did not have sufficient unit discharge to rapidly initiate erosion and subsequent breaking of the clay core. To initiate more rapid erosion, the top of the clay core was lowered to el 688.0 and an impervious membrane was imbedded in the clay core (type 5 design, Plate 19). The membrane extended, on a downstream 45-deg slope, to el 692.0. As overtopping and erosion began, the geomembrane did not rapidly lay down or wash out. The geomembrane was supported on its downstream side by the sand fill. The sand fill was supported by the top of the clay core (Plate 20). This design was resistant to initial failure and required a substantial time (142 min, paragraph 28) for total failure.

33. The type 6 design included an impervious geomembrane sloped upstream (Plate 21) to induce initial failure by erosion of the sand fill on the downstream side of the geomembrane. Also, the clay core was constructed vertically to el 665.0 to determine if a vertical clay core (lower portion) may be more susceptible to failure. Initial flow over the type 6 design scoured the sand fill from the downstream surface of the geomembrane and the geomembrane partially failed, as shown in Plate 22. Partial failure of the geomembrane permitted about a 2.0-ft depth of flow over the eroding top of the fuseplug, which contributed to increasing the rate of failure. Partial support for the geomembrane was provided by the sand fill. The sand fill was supported by the top of the clay core. The vertical portion of the clay core (downstream side) was subjected to increased turbulence and lower pressures, which tended to excavate the supporting granular material at a more rapid rate. The prototype time required for total failure was 57 min (paragraph 28).

34. To reduce support of the geomembrane provided by the clay core, the geomembrane was imbedded in the center of the clay core and extended on the surface of the clay core to its downstream edge and sloped to el 692.0 (type 7 design, Plate 23). The downstream sloping geomembrane in the type 7 design protected the granular material from erosion, and in turn the granular material supported the geomembrane. The prototype time required for total failure was 100 min (paragraph 28).

35. The geomembrane was sloped upstream in the type 8 design (Plate 24) to induce initial failure by erosion of the granular material on the downstream side of the geomembrane. Satisfactory failure of the geomembrane due

to hydrostatic pressure occurred after the covering of granular material was eroded from the surface (downstream side) of the geomembrane. Failure of the geomembrane permitted a flow depth of about 4 ft over the top of the clay core. The prototype time required for total failure was 70 min (paragraph 28). The type 8 design required additional time for total failure due to a large section of the clay core wedging between the sides of the flume.

36. Observation of vertical failure of the clay cores in designs 1-8 revealed that the sloping clay core streamlined the flow and the intensity of turbulence of the downstream side of the clay core was not sufficient to readily excavate the granular material supporting the clay core. The granular material supporting the clay core must first be excavated to permit the clay core to break due to a combination of hydrostatic load and its own weight.

37. The type 9 design (Plate 25) was designed with a vertical clay core to increase the turbulence and thus the rate of hydraulic excavation of the granular material on the downstream side of the clay core. The type 9 design also included the most successful geomembrane design (tested in the type 8 design). Initial flow over the type 9 design scoured the granular material from the downstream surface of the geomembrane, and the geomembrane failed. Failure of the geomembrane permitted about a 4-ft depth of flow over the top of the clay core. The granular material was rapidly excavated from the downstream side of the clay core, and the clay core failed due to hydrostatic pressure. The prototype time required for total failure was 26 min (paragraph 28).

38. The possibility of the geomembrane deteriorating and losing its impermeability was considered a possible detriment to its use. Thus, in lieu of the geomembrane, a 4-ft-diam cylinder (Plate 26) mounted on top of the vertical clay core was investigated (Type 10 design, Plate 27). Satisfactory performance of the cylinder was observed as initial flow over the top eroded the granular material from the downstream side of the cylinder. The cylinder failed and permitted a 4-ft depth of flow over the top of the clay core. Failure of the cylinder is depicted in Plate 27. The prototype time required for total failure of the type 10 design was 19 min (paragraph 28).

Model B

39. Three-dimensional tests were conducted in Model B (scale 1:40) to

further investigate erosion characteristics investigated in two dimensions in Model A. The type 11 design was similar to the type 10 design except for the addition of fixed sheet piles and floating tire wave suppressors to prevent premature overtopping by wave action (Plate 28 and Figure 6). Upstream of the pilot channel, 40-in.-diam tires were used in lieu of sheet piles to reduce the head loss during initial overtopping of the pilot channel.

40. As the type 11 design began to erode laterally, the sand fill on the downstream side of the clay core would stand in a near-vertical position, which reduced the effectiveness of the water in washing away the sand and exposing the clay core. Thus, the sand fill continued to partially support the clay core; therefore, lateral erosion of the spillway was erratic and did not progress as rapidly as planned. Various stages of failure are shown in Photo 2.

41. Following tests of the type 11 design, it was decided by the Nashville District to modify the design by lowering the crest of the pilot channel 0.5 ft to el 691.5 to permit an initial depth of 0.9 ft rather than 0.4 ft in the pilot channel and by changing the fill material on the downstream side of the clay core from a sand to a coarse sand and gravel to reduce the tendency for the fill material to stand near vertically by reducing the capillary and cohesion effects (type 12 design). The reduction in the elevation of the crest of the pilot channel permitted an additional 0.5 ft of head on its crest and thus eliminated the need for the cylinder as a device to trigger failure. Based on observations of three-dimensional failure of the clay core in the type 11 design, WES engineers installed the clay core on a 45-deg slope and reduced its thickness from 50 to 33 percent of the scaled thickness.

42. Typical cross sections through the embankment and pilot channel of the type 12 design are shown in Figure 7 and Plate 29. Wave suppressors were installed as shown in Plate 29. Various stages of failure are shown in Photo 3. After initial flow through the pilot channel (Photos 3a and 3b), about 7 min (prototype) was required for vertical erosion down to the surface of the fixed crest and simultaneous development of about a 75-ft-long breach (Photo 3c). The length of the breach was measured at the surface of the fixed crest. Following formation of the 75-ft-long breach, flow began to laterally erode the spillway (Photo 3d). Lateral erosion on each side of the breach occurred at a rate of 12.5 ft per minute (prototype). The prototype time

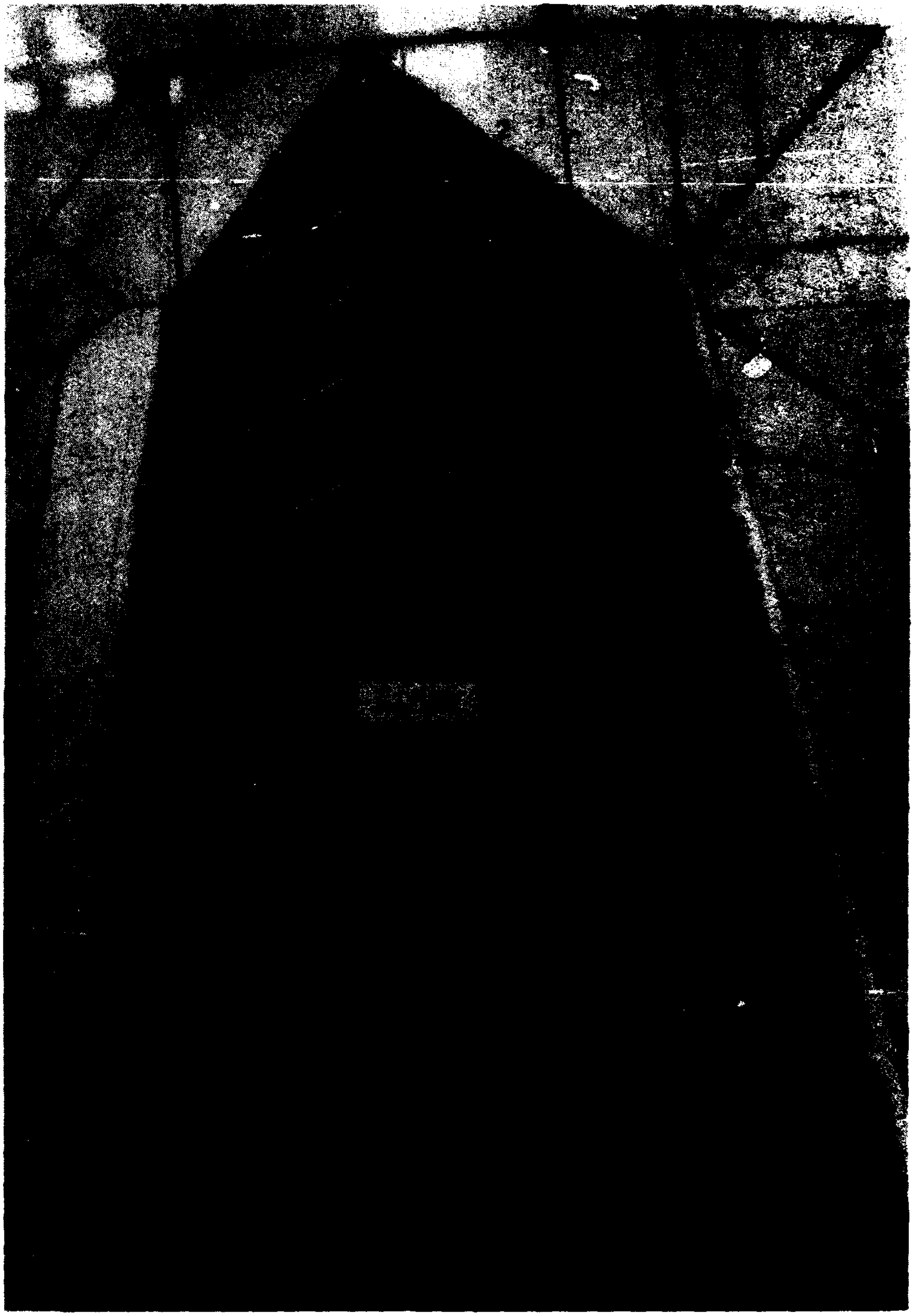

Figure 6. Type 11 design

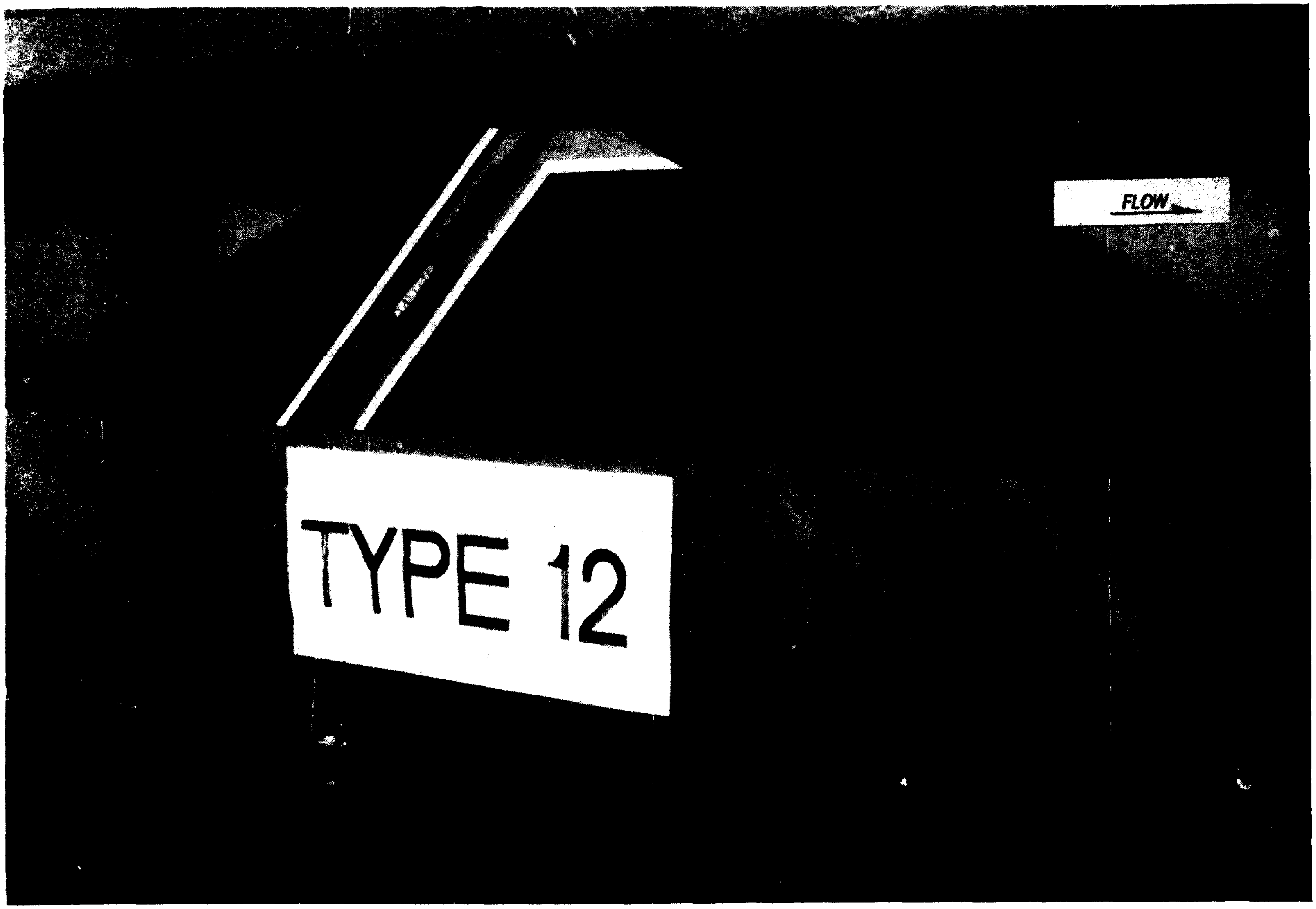

Figure 7. Cross section through pilot channel

required for breaching and failure of the 600-ft-long spillway was 28 min and was computed by the following equation:

$$S_T = B_T + \left[\frac{S_L - B_L}{(2)\ E.R.}\right] \tag{4}$$

where

S_T = time required for spillway failure, min

B_T = time required for breach, min (vertical erosion to surface of fixed crest)

S_L = spillway length, ft

B_L = breach length, ft

E.R. = rate of lateral erosion, ft/min

and thus,

$$S_T = 7\ \text{min} + \left[\frac{600\ \text{ft} - 75\ \text{ft}}{(2)(12.5\ \text{ft/min})}\right] = 28\ \text{min} \tag{5}$$

Model C

43. Various stages of failure in the 1:20-scale model (type 12 design) are shown in Photo 4. Development of a breach about 75 ft long occurred in 7.4 min. Following development of the breach, lateral erosion progressed at a rate of 11.7 ft per minute. Equation 4 indicated that failure of the 600-ft-long spillway would require 29.8 min.

Scale Effects

44. The failure times for the 600-ft-long spillway determined from the 1:40- and 1:20-scale models (Models B and C) were 28 and 29.8 min, respectively, and within 6.5 percent of each other. This is considered good correlation and indicates that scale effects did not significantly affect model results. Based on the average of the times required for failure determined in the two models, the time predicted for failure of the prototype fuseplug spillway is 28.9 min ± 0.9 min.

Discharge Coefficients and Velocities

45. Tests were conducted in the 12-ft-wide flume at a scale of 1:40 (Model B) to determine discharge coefficients for the 600-ft-long fixed crest.

46. To determine the crest coefficients, a 480-ft-long section of the fixed spillway with no abutments was simulated, as shown in Figure 8.

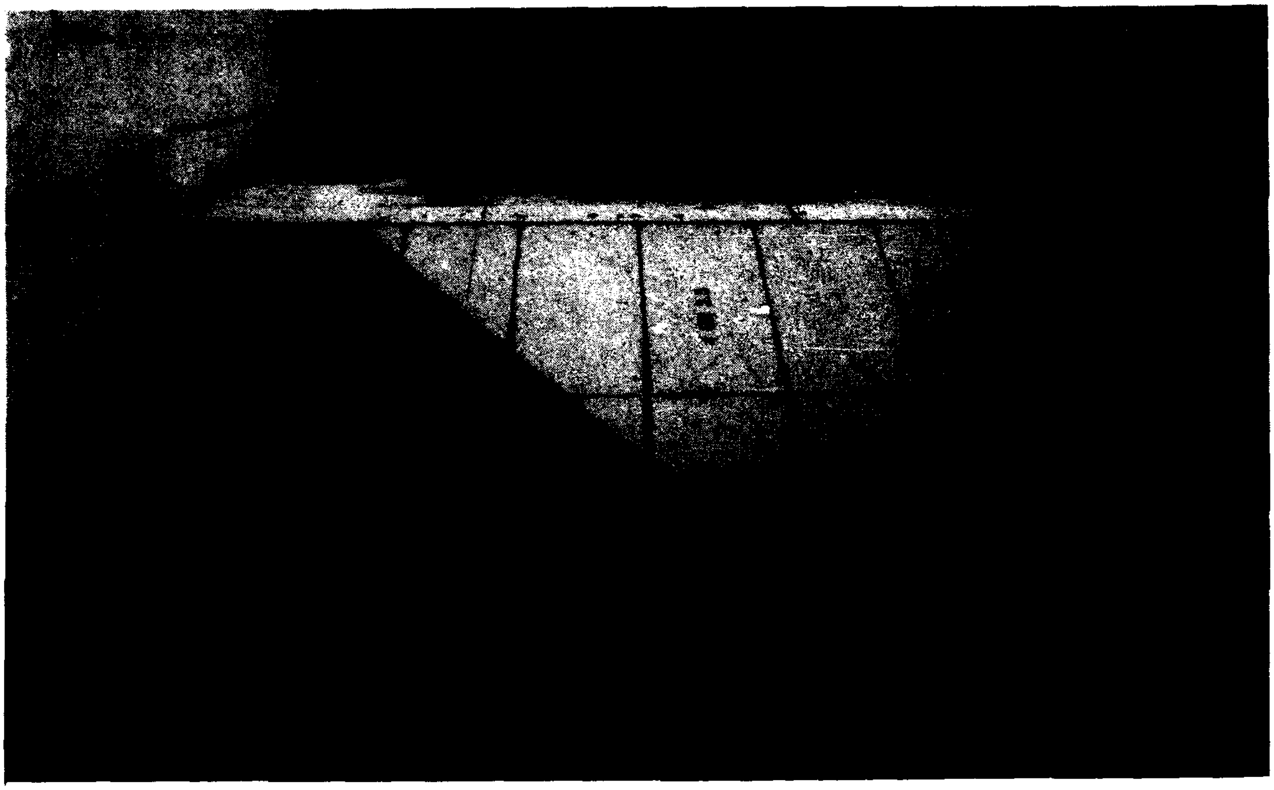

Figure 8. Model B, fixed crest

Spillway calibration data were obtained by setting various discharges and for each discharge measuring the head on the broad-crested spillway. The head on the crest was measured 300 ft upstream from the lateral center line of the crest. The energy heads generated by the approach velocities were negligible and were not added to the measured heads on the crest. The discharge coefficients were computed by the following empirical equation:

$$C = \frac{Q}{LH^{3/2}} \tag{6}$$

where

C = discharge coefficient

Q = discharge, cfs

L = net crest length, ft = 480 ft

H = head on crest, ft

Discharge coefficient versus head on the crest is shown in Plate 30. The basic data obtained from the model are listed in the following tabulation:

Q cfs	H ft	C
25,300	8.60	2.09
45,500	12.32	2.19
45,500	12.48	2.16
70,800	15.44	2.43
88,000	17.68	2.47
105,200	19.40	2.57
126,500	21.76	2.60
136,600	22.72	2.63
164,900	25.04	2.74
191,250	27.00	2.84
207,400	28.64	2.82
242,900	31.28	2.89
262,100	33.24	2.85
283,300	34.96	2.86
303,600	35.92	2.94

47. To determine the abutment contraction coefficients, the right abutment was installed (Figure 9). Simulation of the right abutment in the model reduced the net length of the simulated crest to 400 ft. Spillway calibration data were obtained by the same method exercised with only the crest simulated. The abutment contraction coefficients were obtained by the following empirical equation:

$$Q = C\left[L - 2(K_a)H\right]H^{3/2} \tag{7}$$

or

Figure 9. Model B, fixed crest and right abutment

$$K_a = \frac{CLH^{3/2} - Q}{CH^{5/2}} \tag{8}$$

where

K_a = abutment contraction coefficient

C = discharge coefficient (Plate 30)

L = 400 ft

The abutment contraction coefficients versus head on the crest are plotted in Plate 31. The design of the left abutment is identical to that of the right abutment. Therefore, the abutment contraction coefficient is assumed to be the same at each abutment. The abutment contraction coefficients in Plate 31 are negative because the model tests indicate that the loss in discharge due to flow contraction at the abutments is more than compensated for by the increase in flow area provided by the 1V on 2H sloped abutments. The basic data are listed in the following tabulation:

Q cfs	H ft	C	K_a
291,400	37.20	2.59	−0.95
253,000	34.28	2.92	−0.92
227,700	32.00	2.90	−1.05
200,400	29.32	2.85	−1.46
149,800	25.20	2.74	−1.28
124,500	22.80	2.65	−1.38
93,100	19.40	2.52	−1.67
52,600	14.16	2.33	−1.67
37,400	11.60	2.22	−2.28

48. A sample calculation to determine discharge for the Center Hill fixed crest for the design head on the crest of 34.4 ft is illustrated in Equations 9–13.

$$C = 2.92 \text{ (Plate 30)} \tag{9}$$

$$K_a = -0.99 \text{ (Plate 31)} \tag{10}$$

$$L = 600 \text{ ft} \tag{11}$$

$$Q = C[L - 2(K_a)H]H^{3/2} \tag{12}$$

$$Q = 393{,}000 \text{ cfs} \tag{13}$$

49. Tests with the fixed crest and right abutment simulated at a scale of 1:40 (Model B) were conducted to measure the magnitude and direction of velocities along the right side of the exit area. The velocities were measured 2 ft above the bottom with a pitot tube during a simulated unit discharge equivalent to 393,000 cfs passing through the prototype spillway at a head on the spillway crest of 34.4 ft (Plate 32).

PART V: DISCUSSION AND SUMMARY OF TEST RESULTS

50. Tests were performed by the Geotechnical Laboratory to characterize the soils of which the prototype fuseplug material is to be constructed to produce materials for use in the scale models.

51. Three physical models were used to develop a satisfactory fuseplug spillway design that would fail at a prescribed rate. Two-dimensional tests conducted at a 1:40 scale in the 1-ft-wide section model (Model A) permitted a qualitative evaluation of vertical failure characteristics of the fuseplug spillway. Tests in Model A were directed primarily toward developing a design that would readily permit initiation of failure by overtopping. Initially a depth of flow of only 0.4 ft was permitted through the pilot channel. This shallow depth did not readily induce initial erosion in the pilot channel. Devices such as a geomembrane or cylinder mounted on the clay core were effective for triggering erosion of the pilot channel. Tests in the two-dimensional model also indicated that vertical failure of the spillway occurred more rapidly with a vertical rather than sloping clay core.

52. Three-dimensional tests conducted at a 1:40 scale in the 12-ft-wide flume (Model B) revealed the need for lowering the invert of the 50-ft-long pilot channel by 0.5 ft to permit an initial depth of flow of 0.9 ft rather than 0.4 ft. This additional depth eliminated the need for devices to trigger failure. Model B indicated that lateral failure of the fuseplug spillway would be satisfactory with a sloping clay core.

53. About 7 min (prototype) was required for flow through the pilot channel to erode vertically down to the surface of the fixed crest and open a 75-ft-long breach (measured at the surface of the fixed crest). Lateral erosion on each side of the breach occurred at a rate of 12.5 ft per minute (prototype). Breaching and failure of the 600-ft-long spillway required 28 min.

54. Three-dimensional tests were conducted in the 12-ft-wide flume (Model C) at a 1:20 scale to determine if scale effects had a significant influence on hydraulic performance. The spillway failed in 29.8 min in Model C. The failure times of the 600-ft-long spillway computed from results obtained from Models B and C are within 6.5 percent of each other. This is good correlation and indicates that scale effects did not significantly affect model results.

55. Fixed crest discharge coefficients were computed based on test

results to determine the crest and abutment contraction coefficients.

56. The magnitude and direction of currents along the right side of the exit area were measured during the maximum anticipated discharge.

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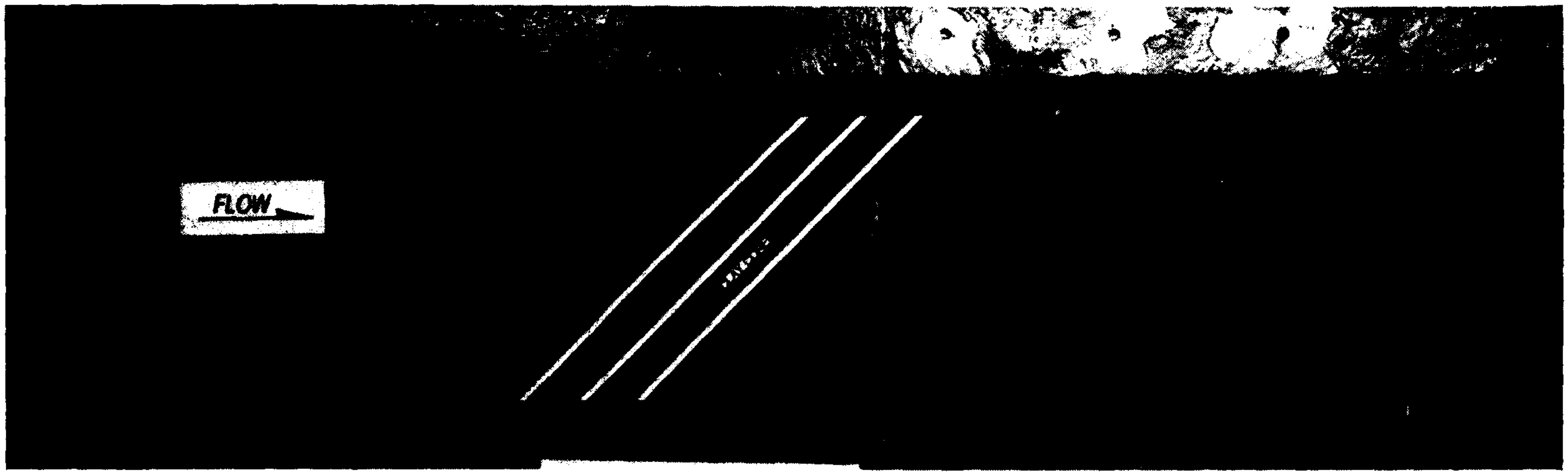
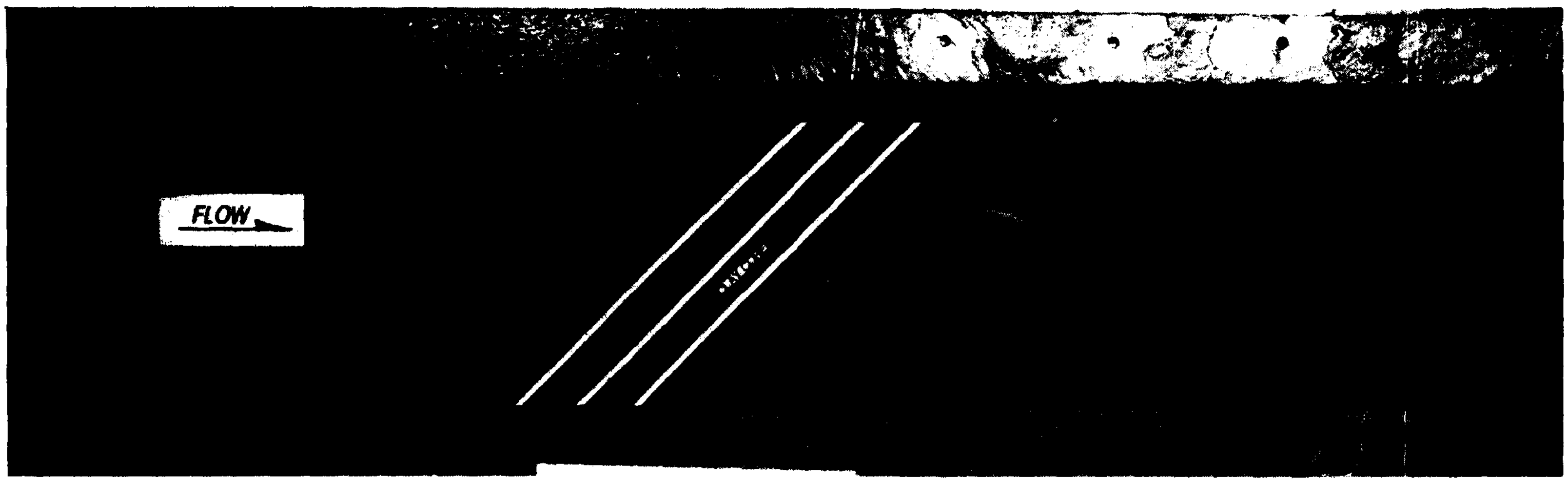
Tinney, E. Roy, and Hsu, H. Y. 1961. "Mechanics of Washout of an Erodible Fuse Plug," Journal of the Hydraulics Division, Proceedings, ASCE, Vol 87, No. HY3, pp 1-29.

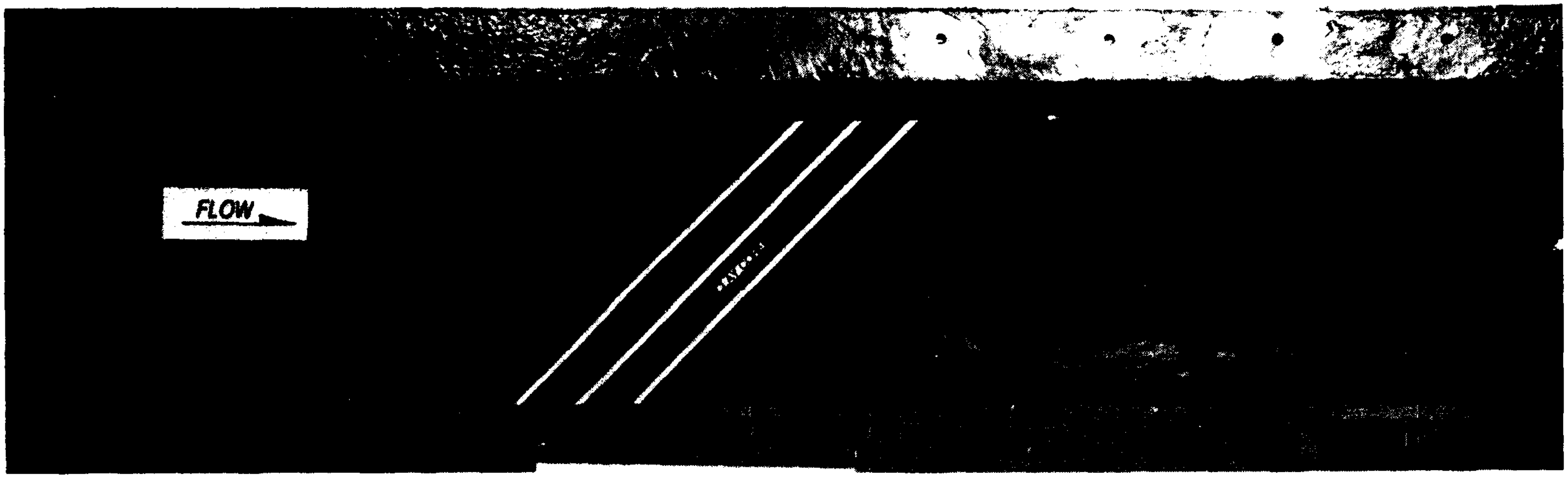


a. Stage 1

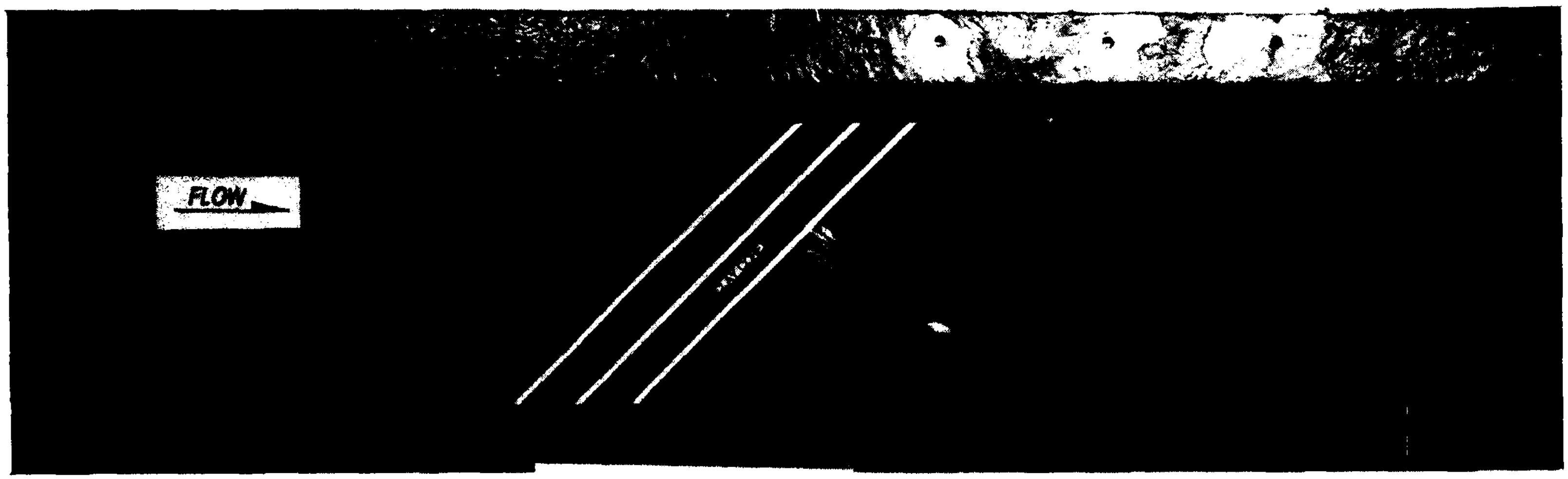


b. Stage 2

Photo 1. Model A, type 3 design, stages of failure (Continued)

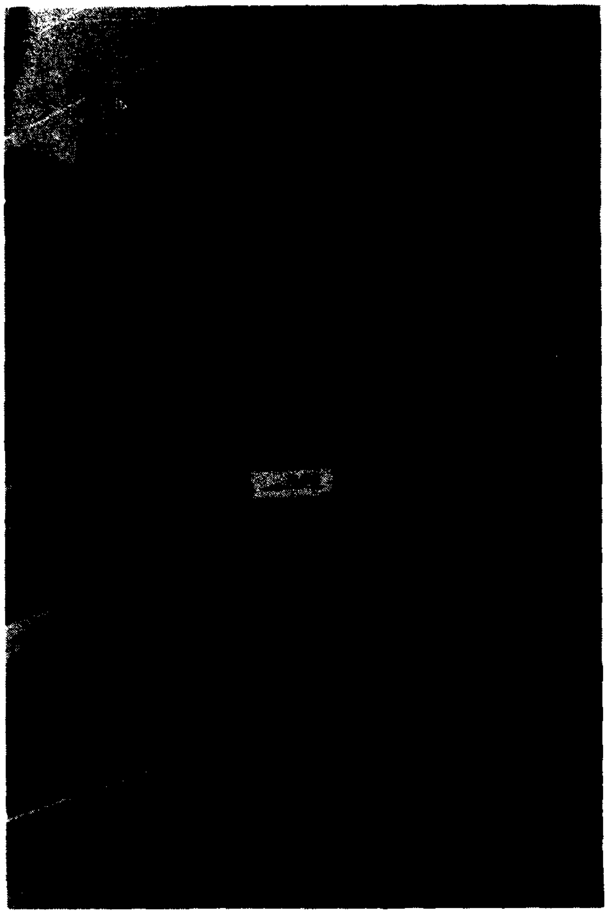


c. Stage 3

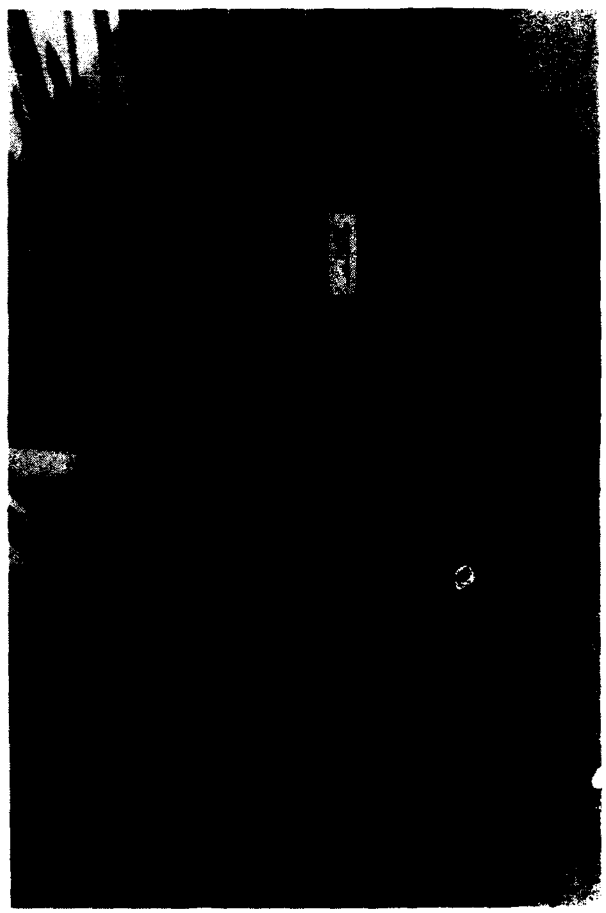


d. Stage 4

Photo 1. (Concluded)

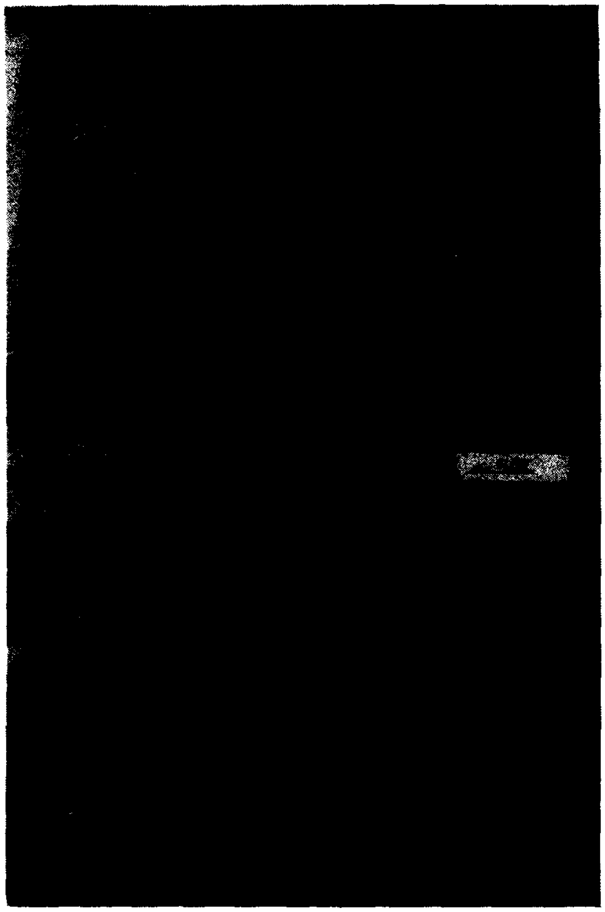


a. Initial overtopping

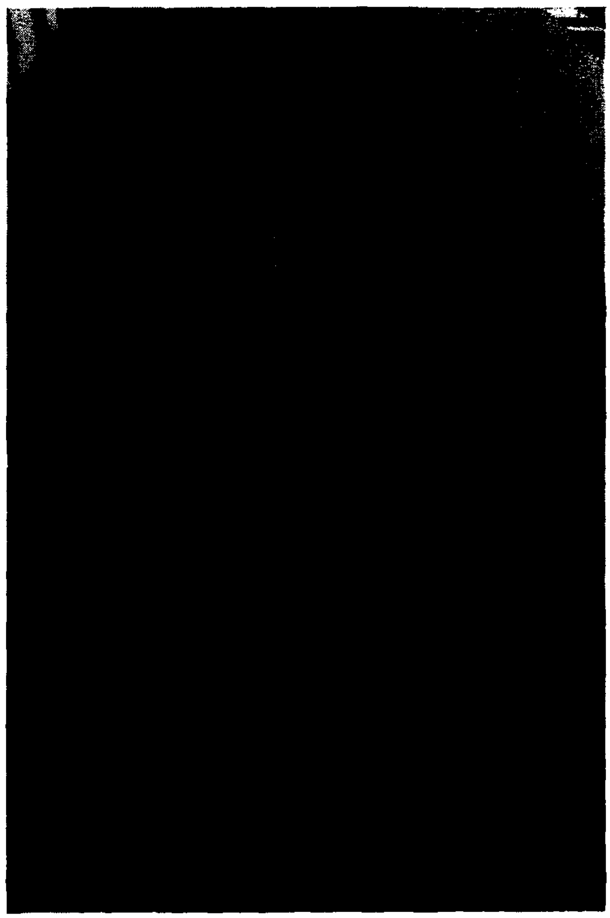


b. Initial cylinder displacement

Photo 2. Stages of failure, type 11 design (Continued)



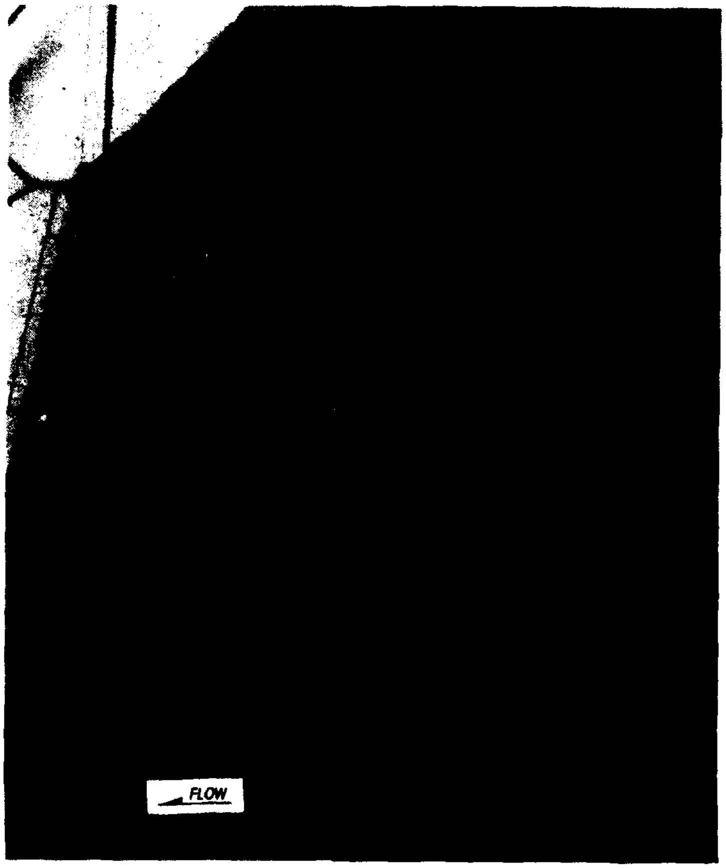
c. Profile of cylinder displacement



d. Vertical erosion

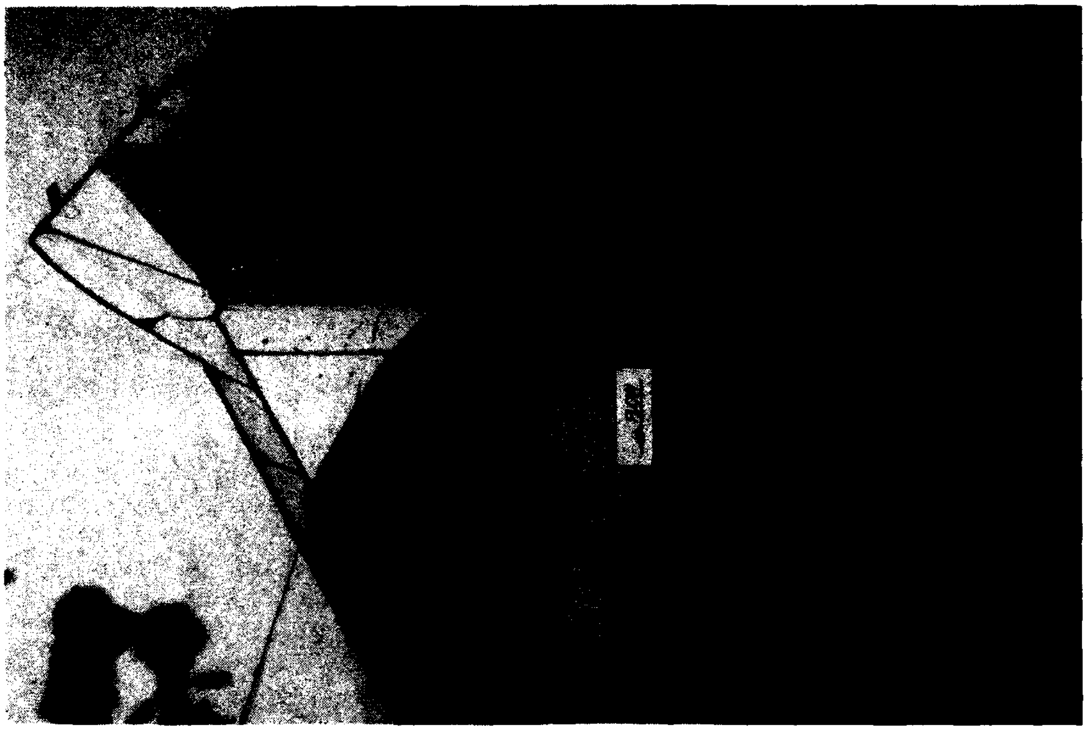
Photo 2. (Concluded)

a. Initial flow through pilot channel

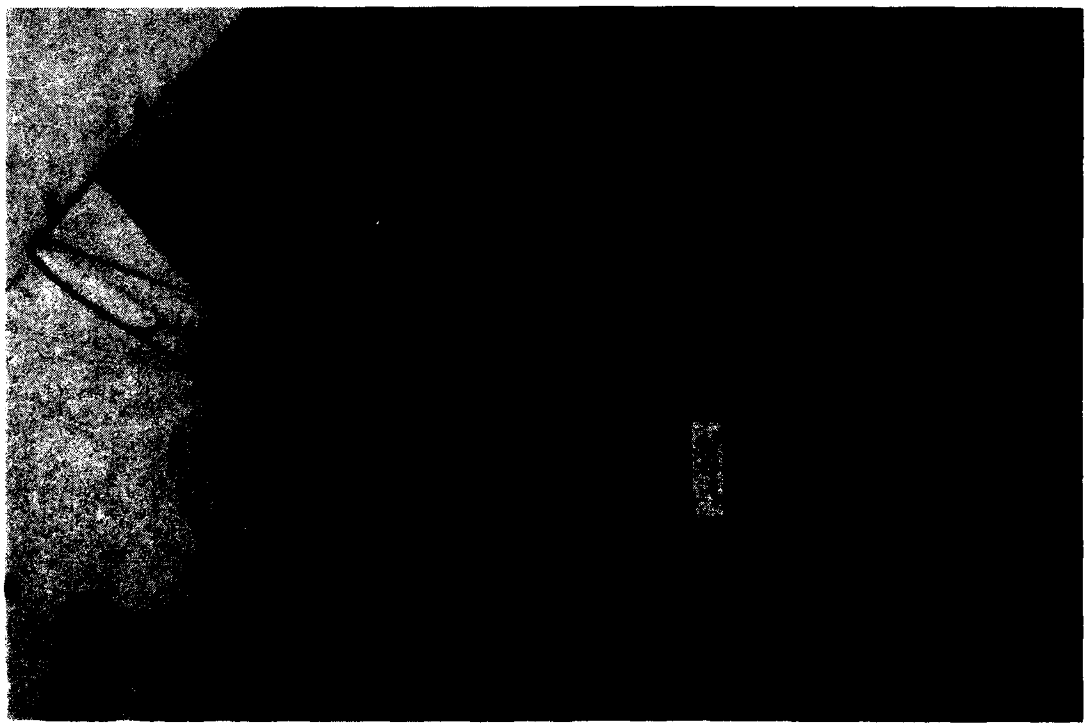


b. Initial flow through pilot channel
(side view)

Photo 3. Stages of failure, type 12 design (Sheet 1 of 3)

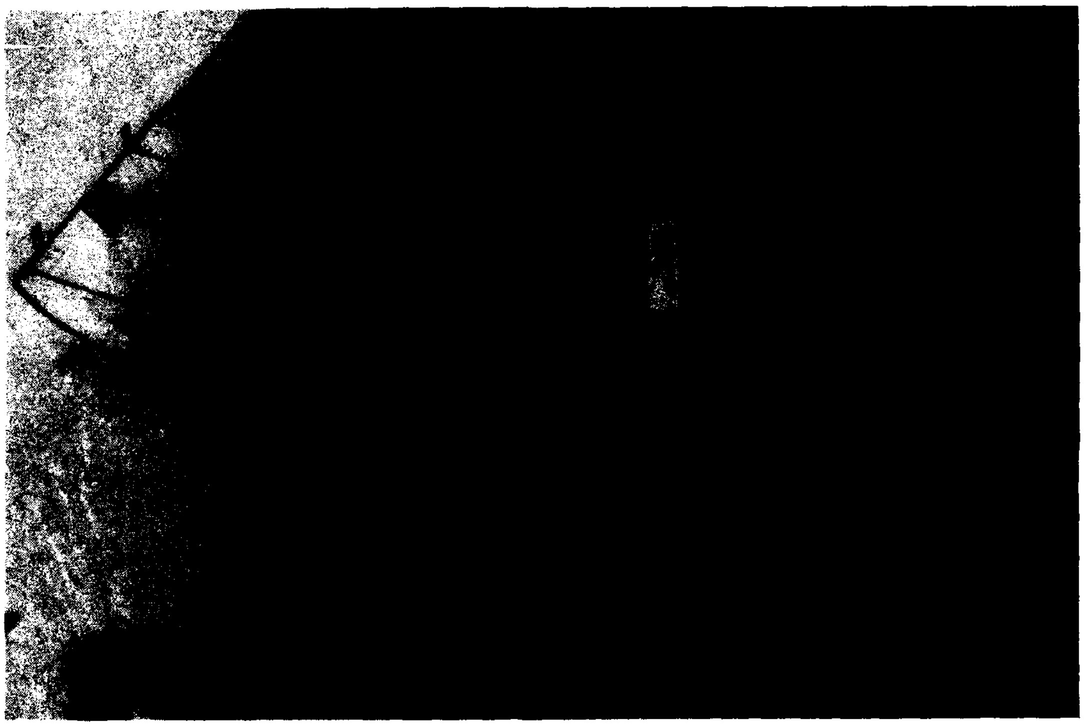


c. 75-ft-long breach



d. Lateral erosion

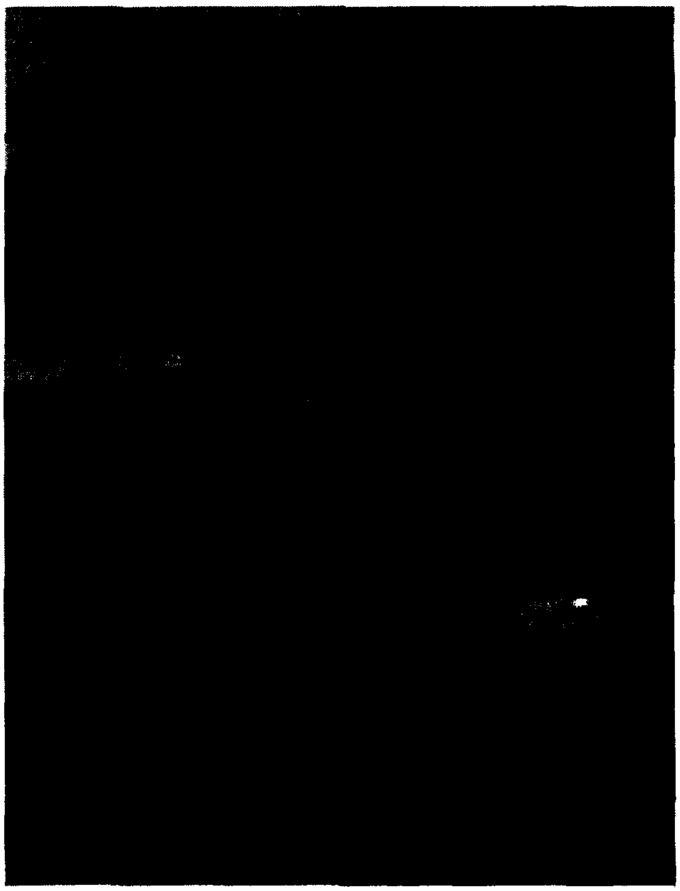
Photo 3. (Sheet 2 of 3)



e. Complete erosion of spillway

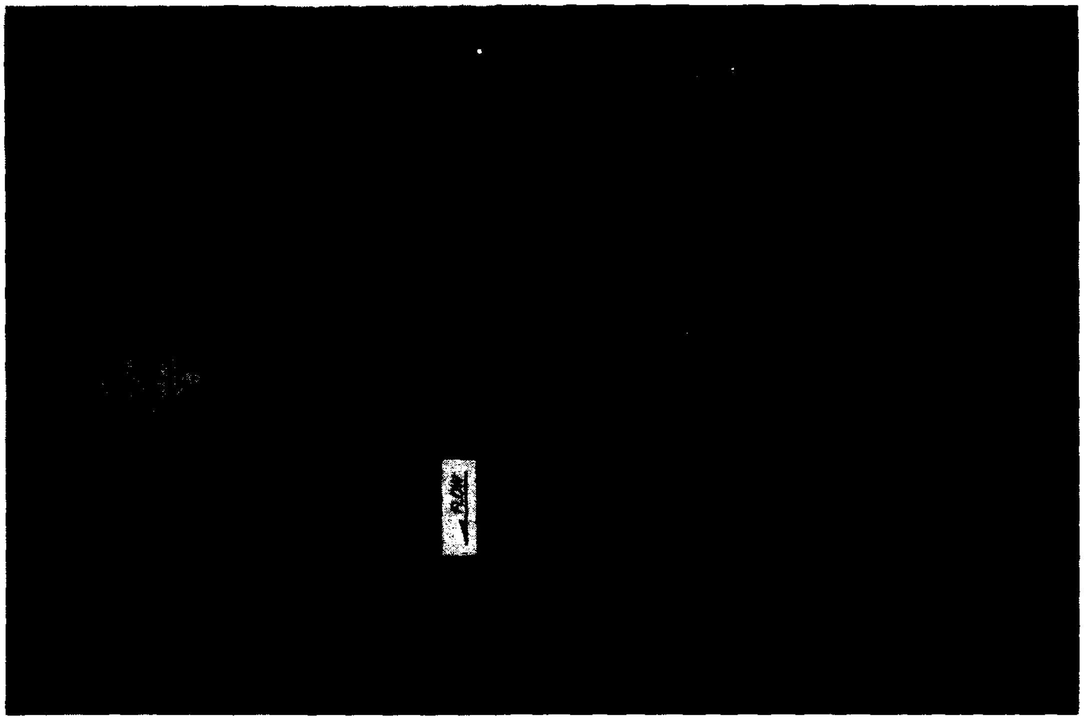
Photo 3. (Sheet 3 of 3)

a. Initial flow through pilot channel

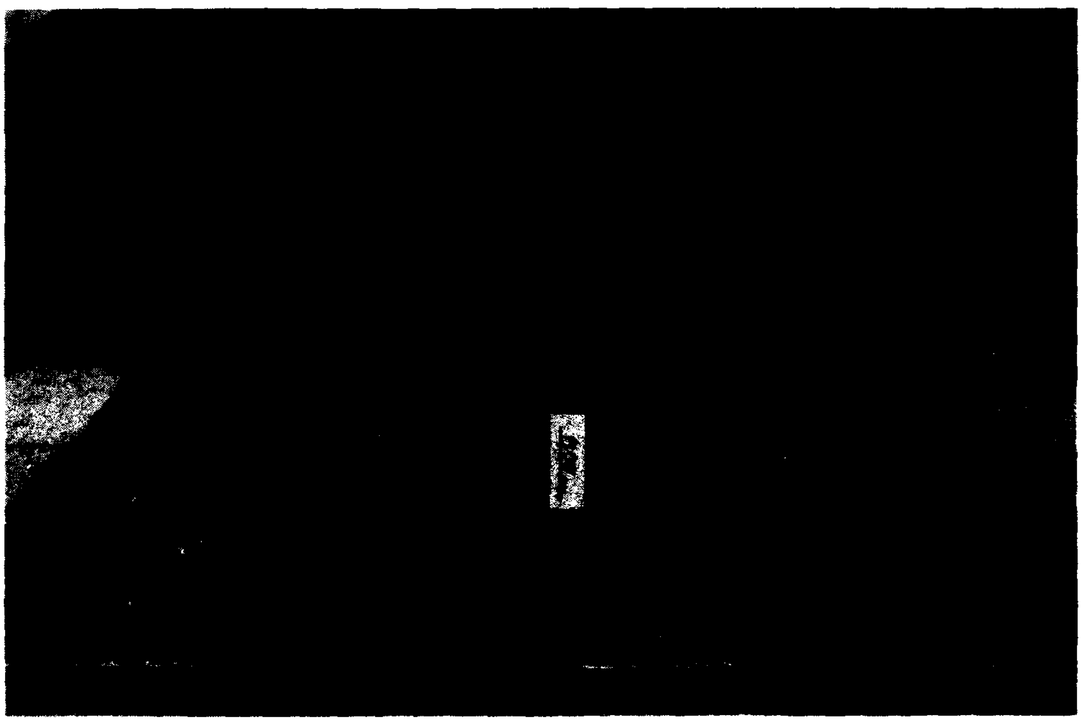


b. Initial flow through pilot channel
(side view)

Photo 4. Stages of failure, type 12 design (Model C) (Continued)

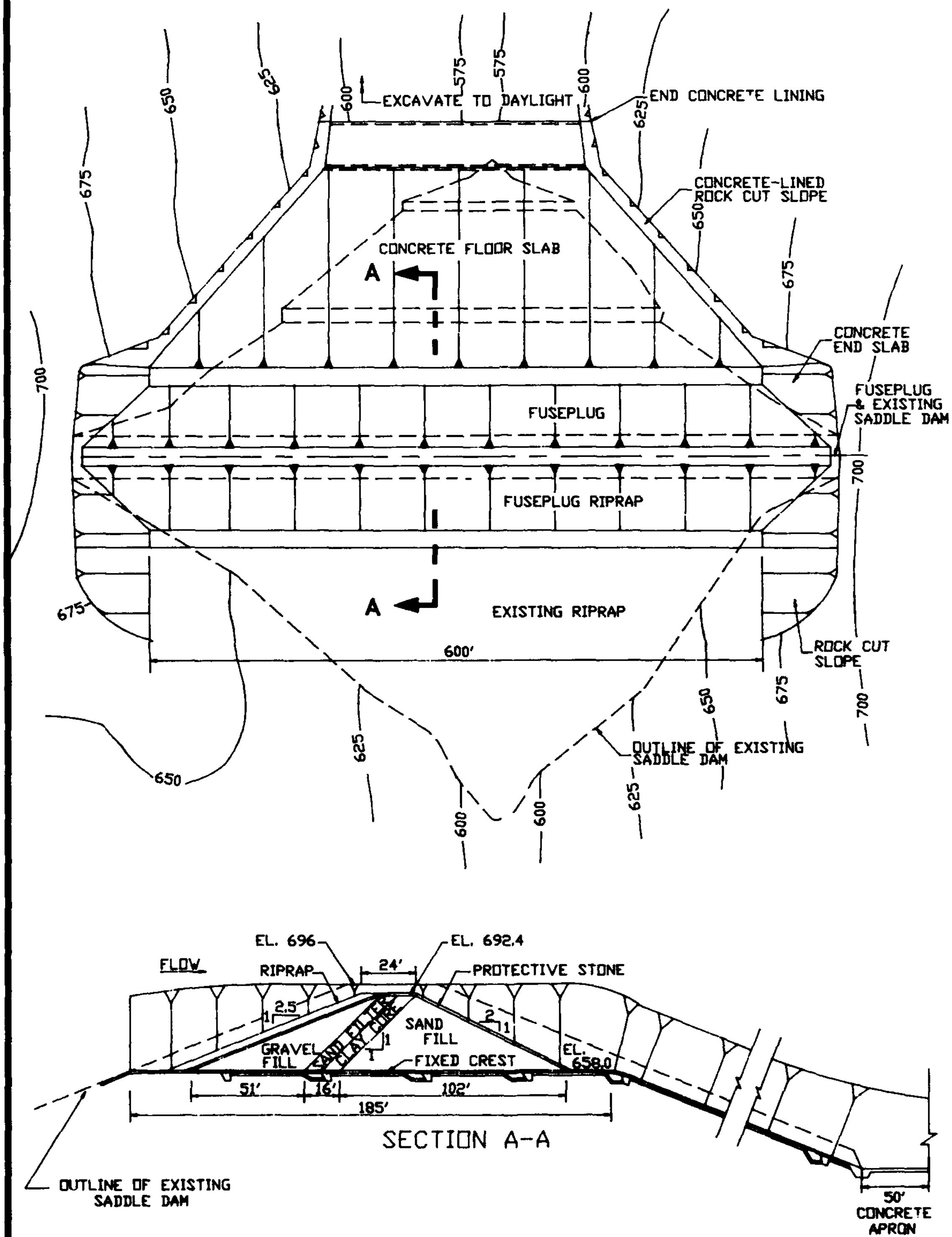


c. 75-ft-wide breach



d. Lateral erosion

Photo 4. (Concluded)



PLAN VIEW AND TYPICAL SECTION

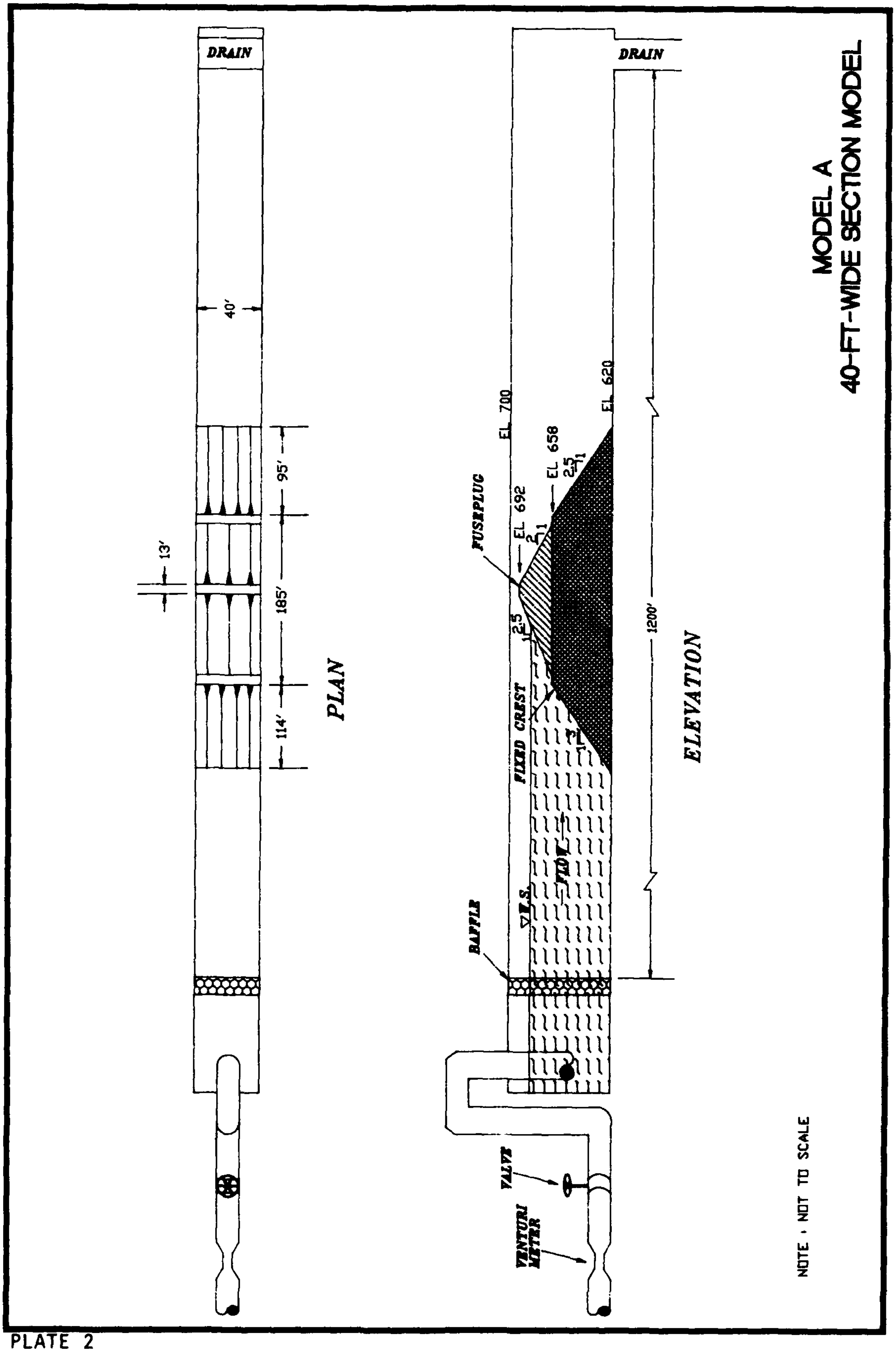
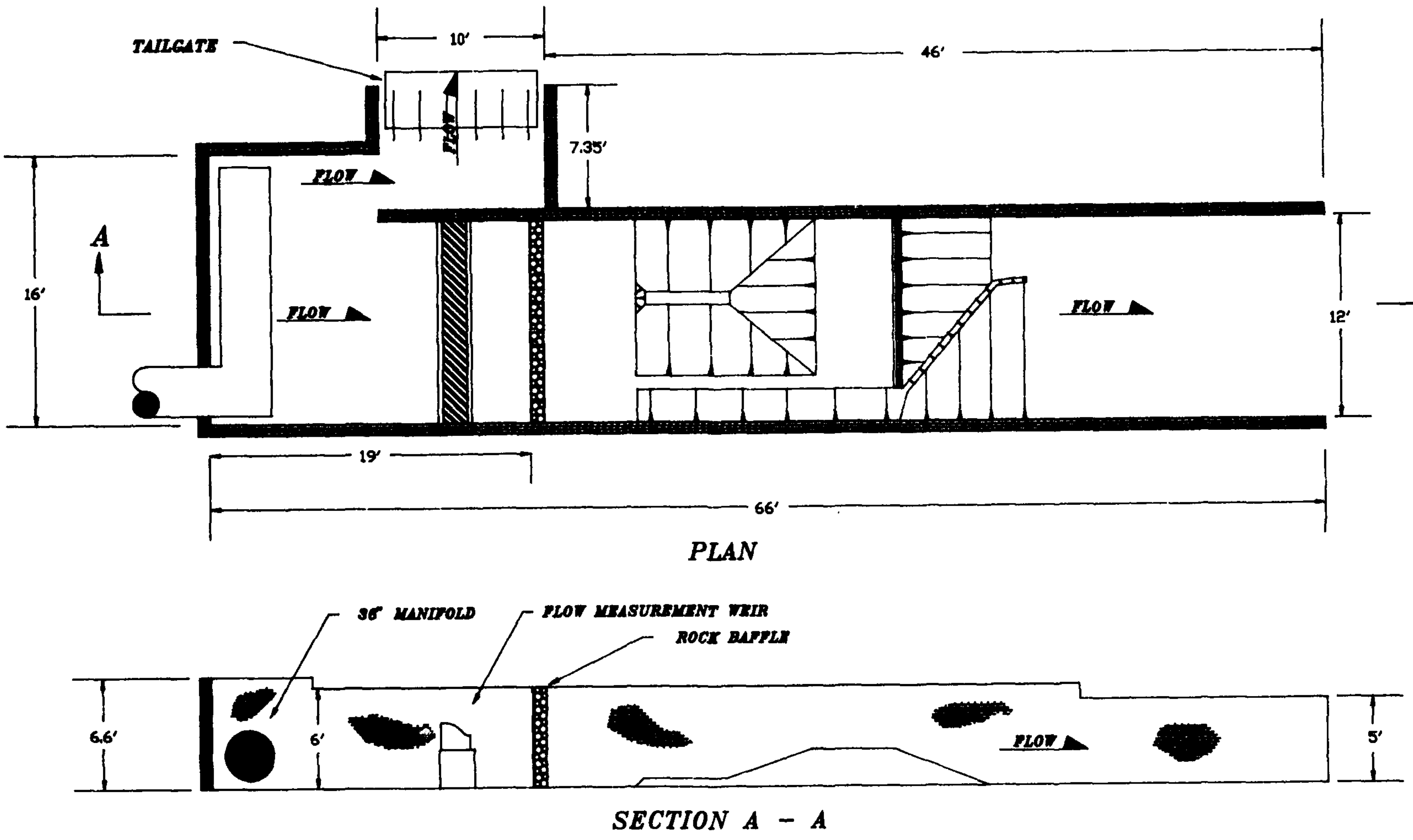
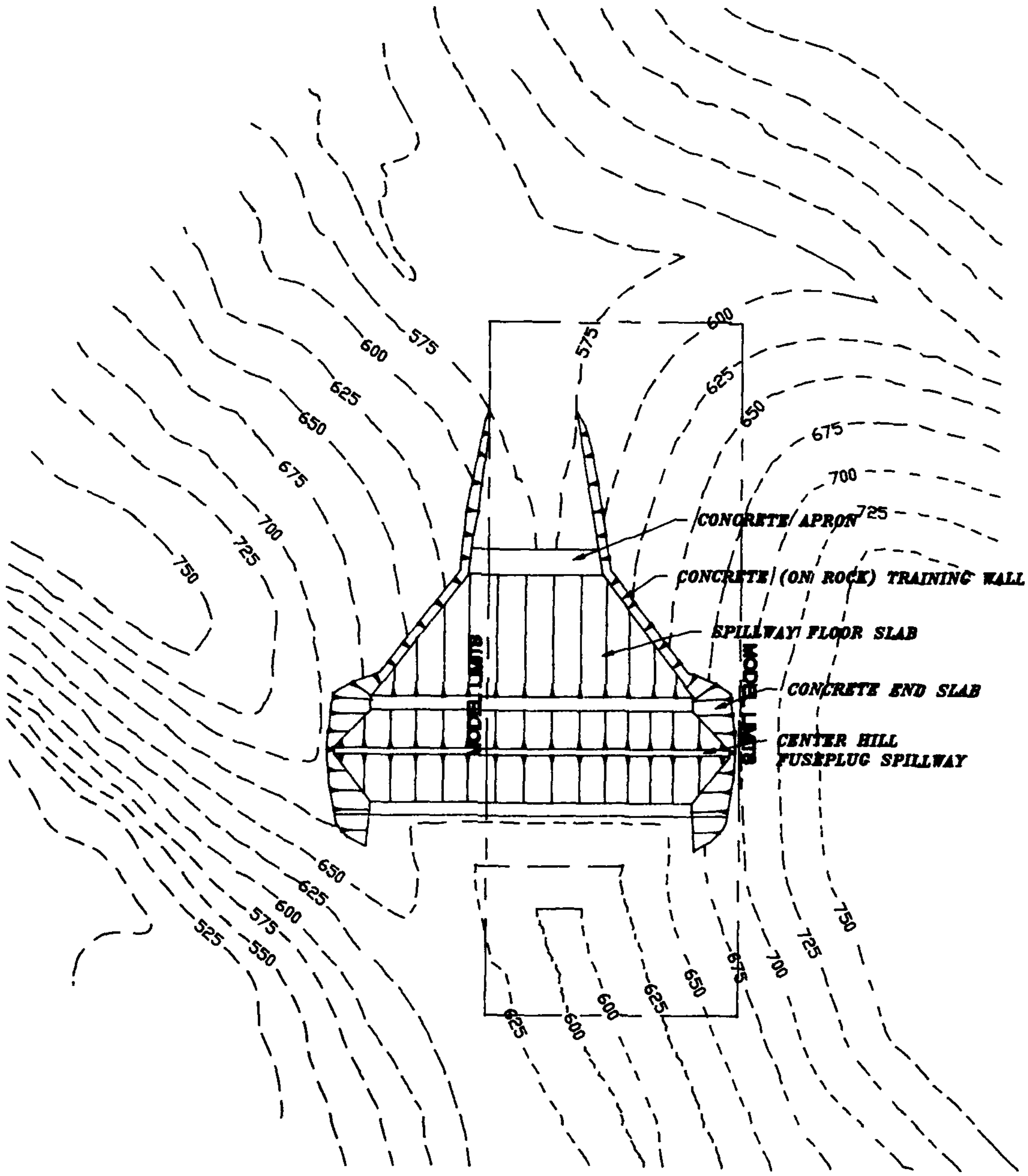

MODEL A
40-FT-WIDE SECTION MODEL
DRAIN
40'
13'
95'
185'
114'
PLAN
DRAIN
EL 700
EL 620
EL 658
EL 692
FUSEPLUG
FIXED CREST
FLOW
W.S.
BAFFLE
1200'
ELEVATION
VALVE
VENTURI METER
NOTE : NOT TO SCALE

PLATE 2

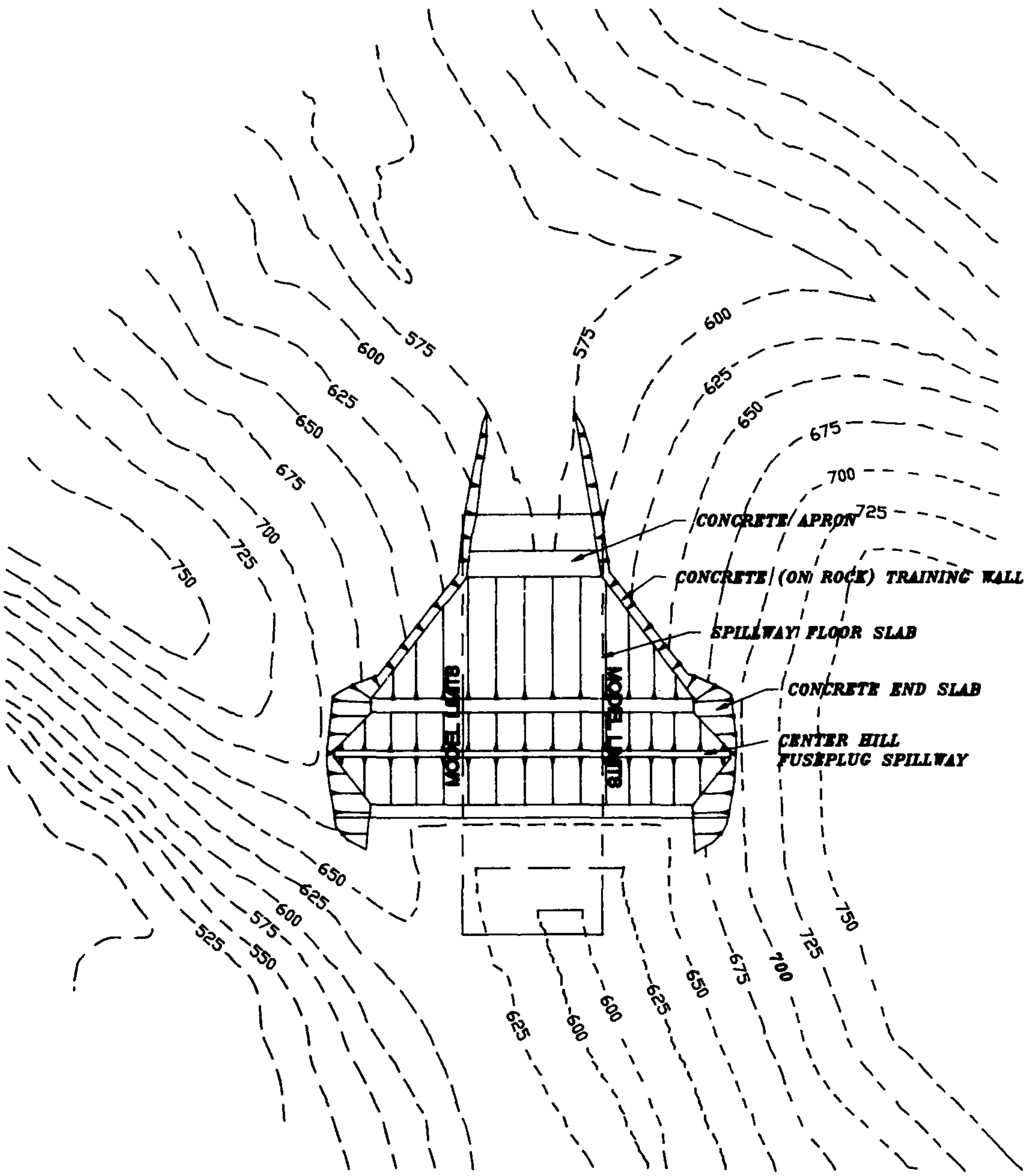


MODEL B
480-FT-WIDE SECTION MODEL

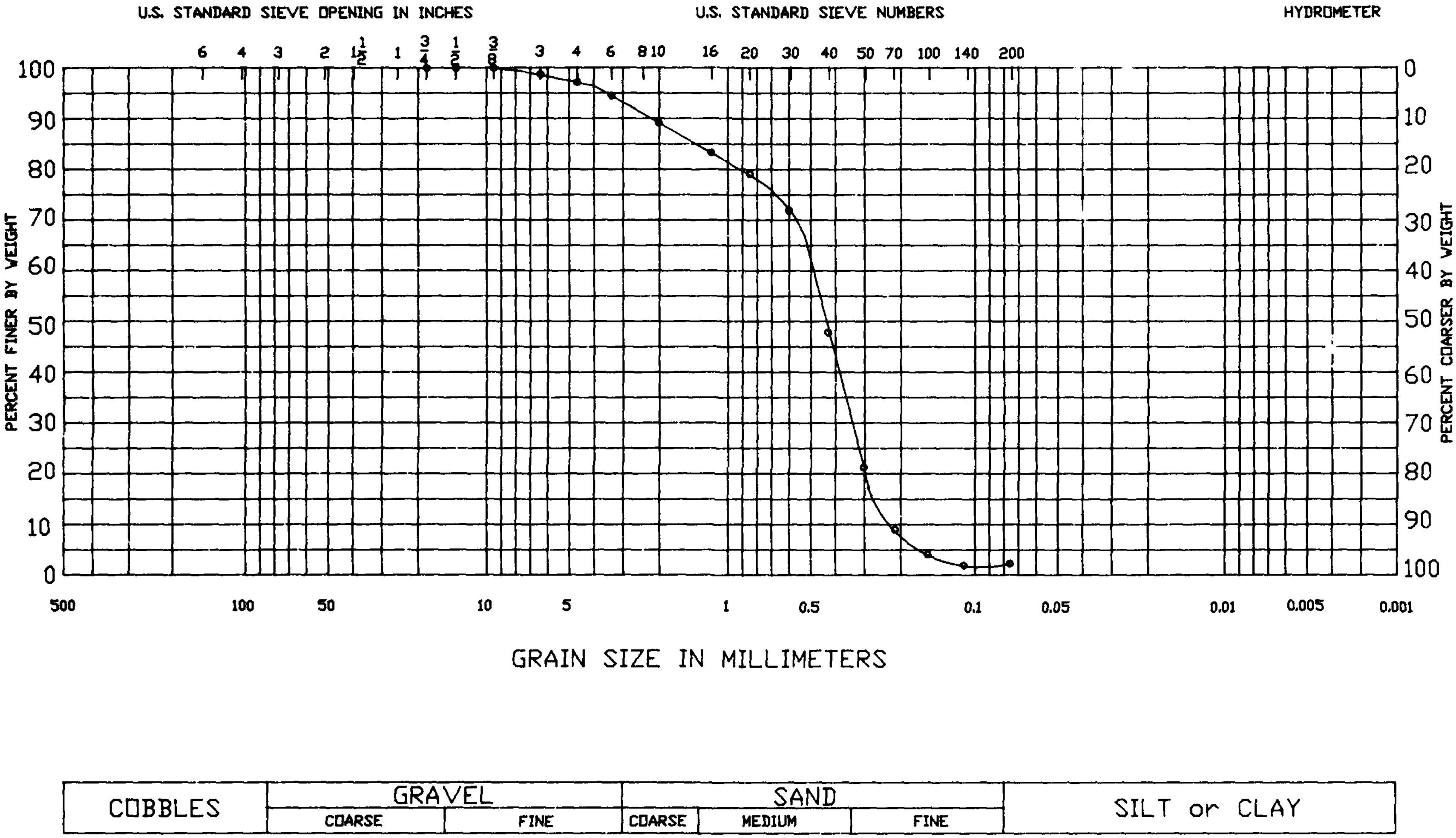


GENERAL PLAN
MODEL B
MODEL SCALE 1:40

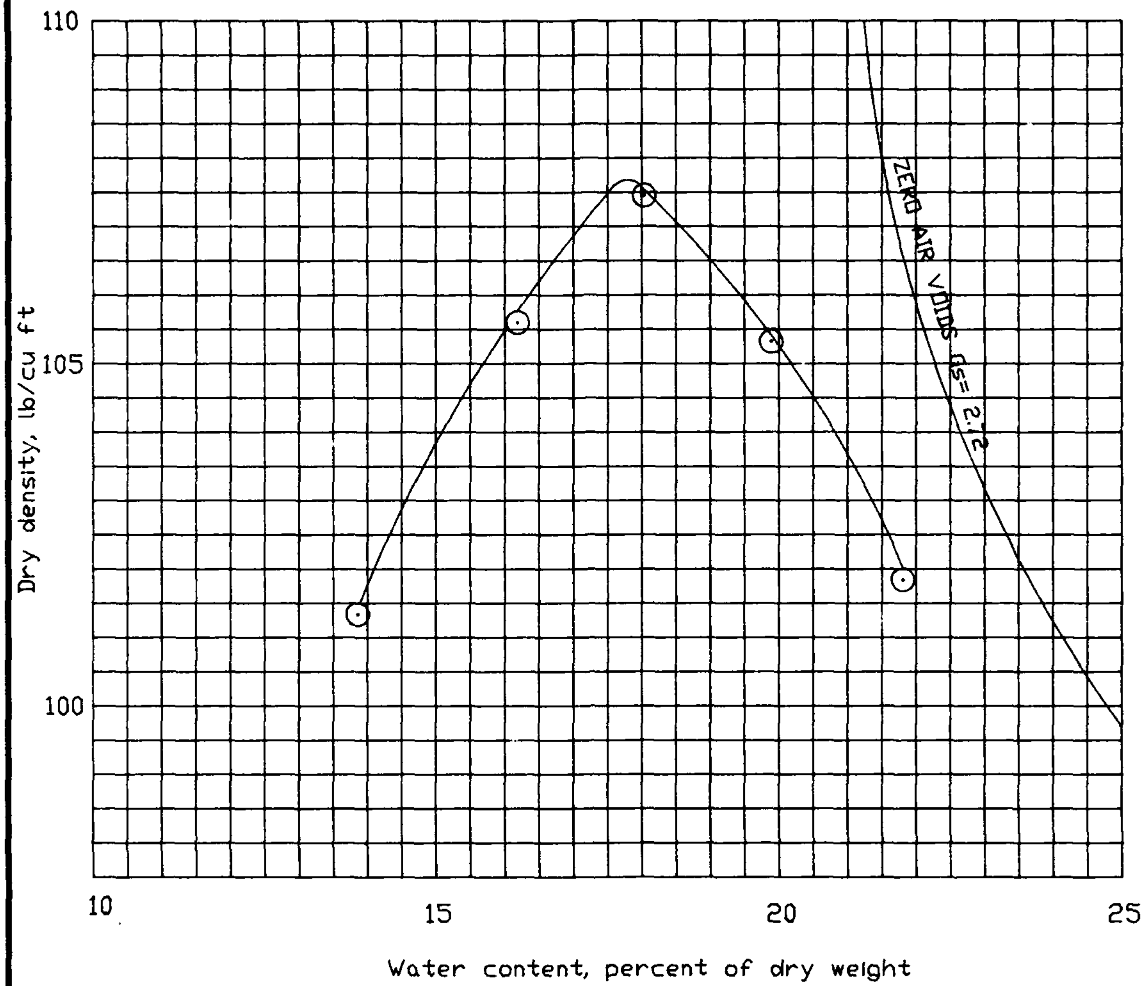
PLATE 4



GENERAL PLAN
MODEL C
MODEL SCALE 1:20



GRAIN - SIZE DISTRIBUTION OF SAND FILL MATERIAL



Standard compaction test

(sleeve)

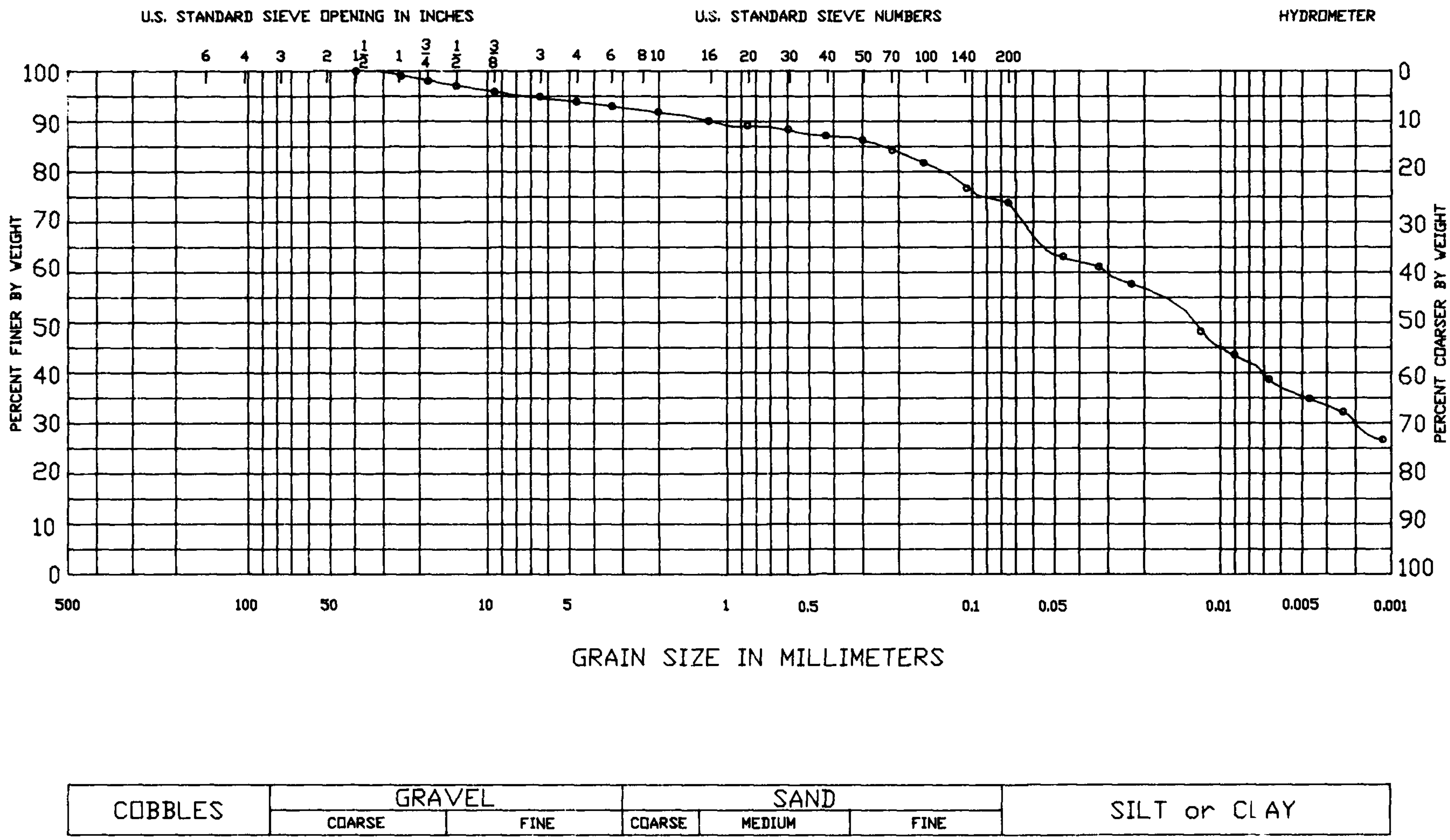
56 blows per each of 3 layers, with 5.5 lb rammer and

12 inch drop. 6 inch diameter mold

Sample No.	Elev or Depth	Classification	Gs	LL	PL	% > NO. 4	% > 3/4 in.
		SANDY CLAY (CL), BROWN, WITH GRAVEL	2.72	42	15	5.5	1.1

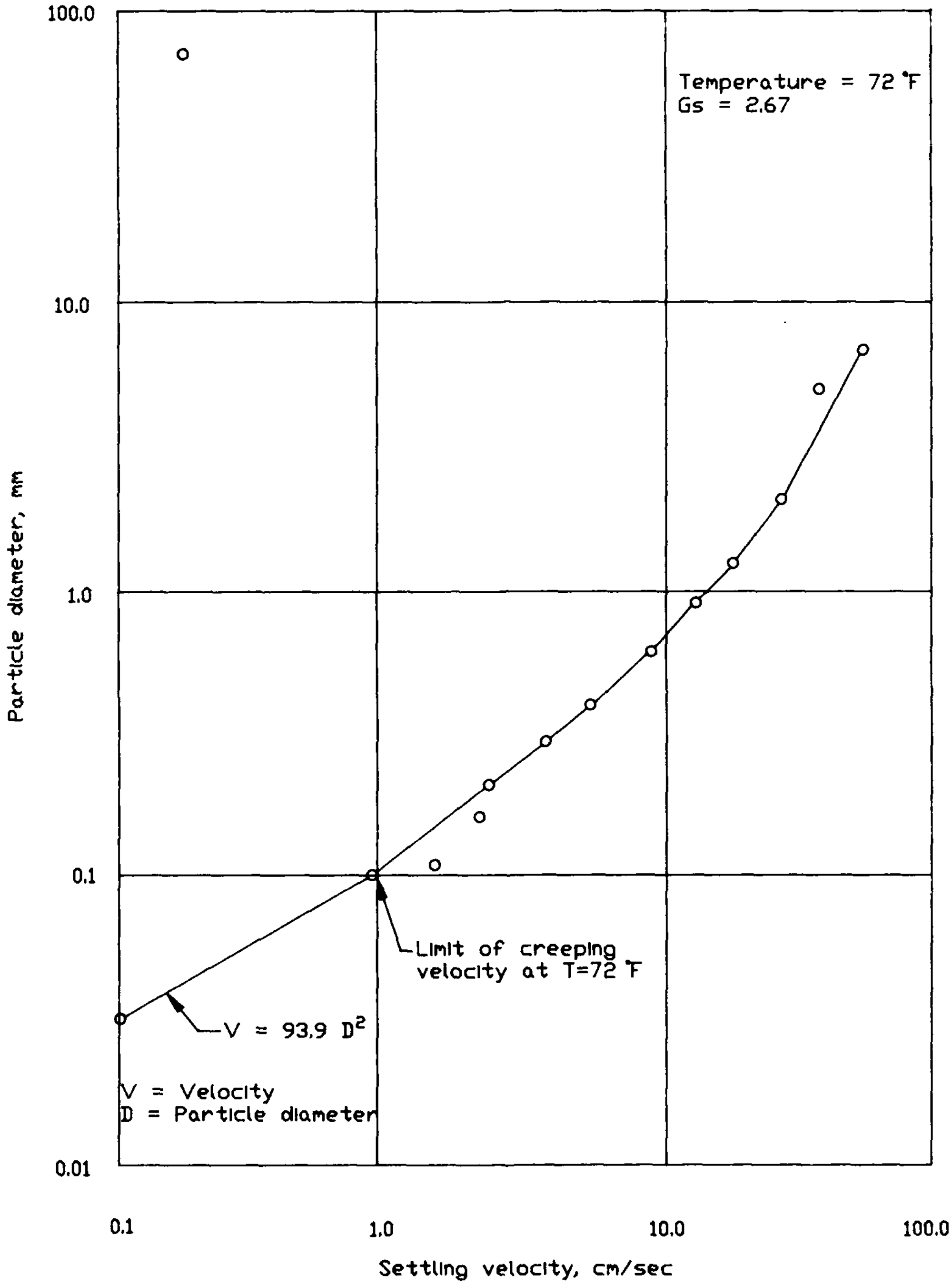
Sample No.				
Natural water content, percent				
Optimum water content, percent				
Max dry density, lb/cu ft				

Remarks	Project
	Area
	Boring No. CLAY (BAGS 1-3) Date 30JAN91
	COMPACTION TEST REPORT

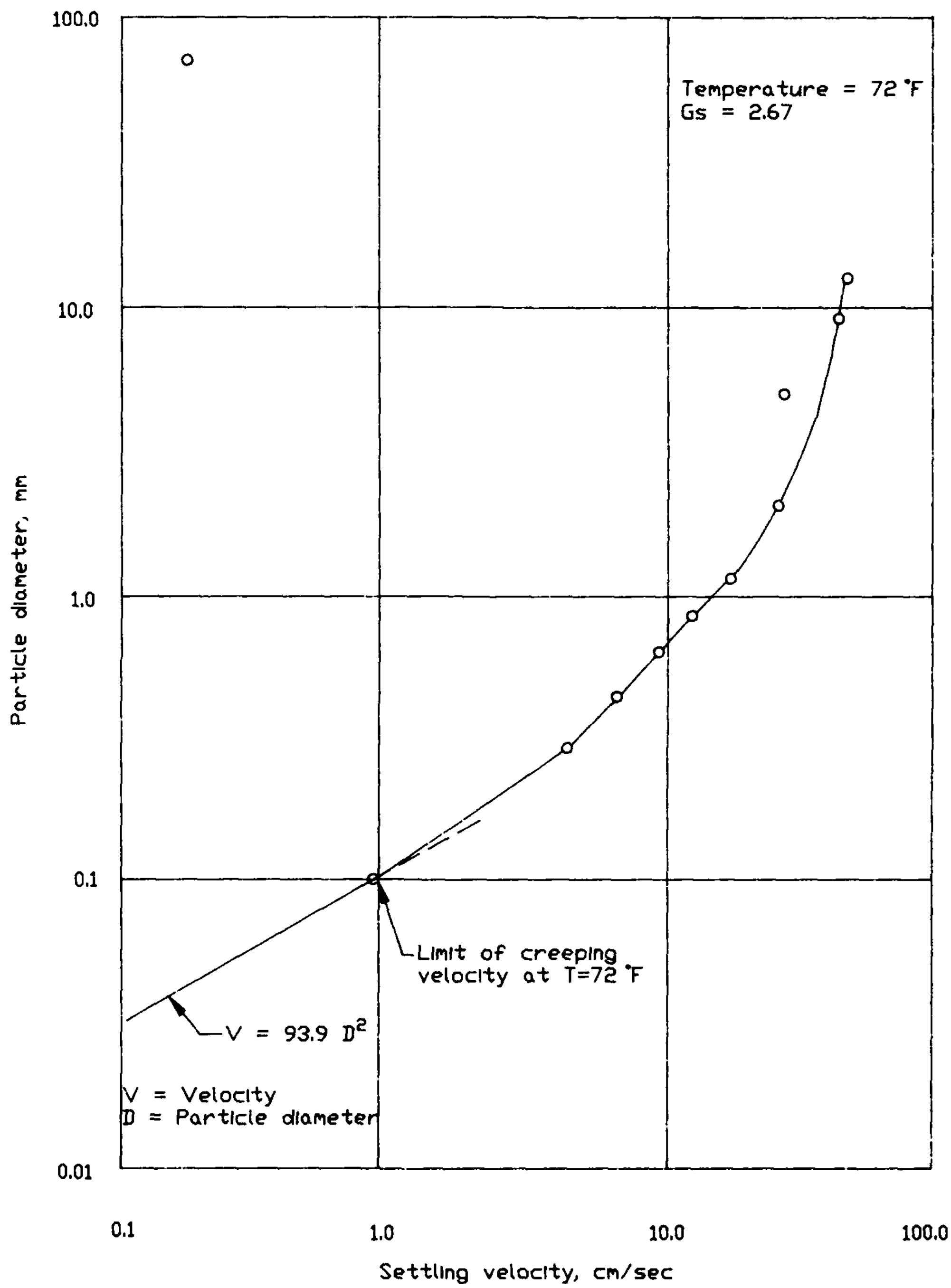


GRAIN - SIZE DISTRIBUTION OF CLAY

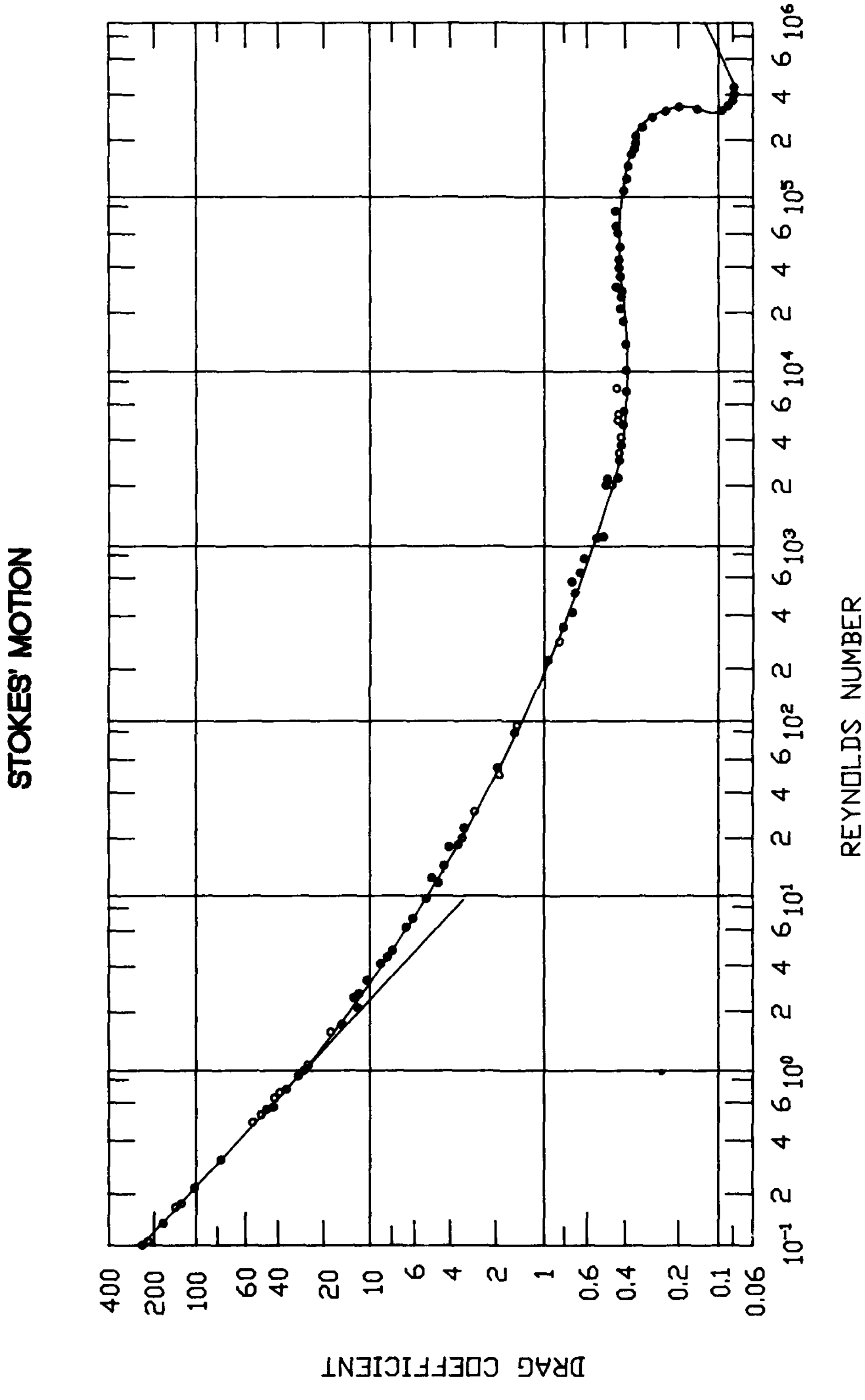
PLATE 8



SETTLEMENT VELOCITY VERSUS PARTICLE DIAMETER SAND FILL MATERIAL

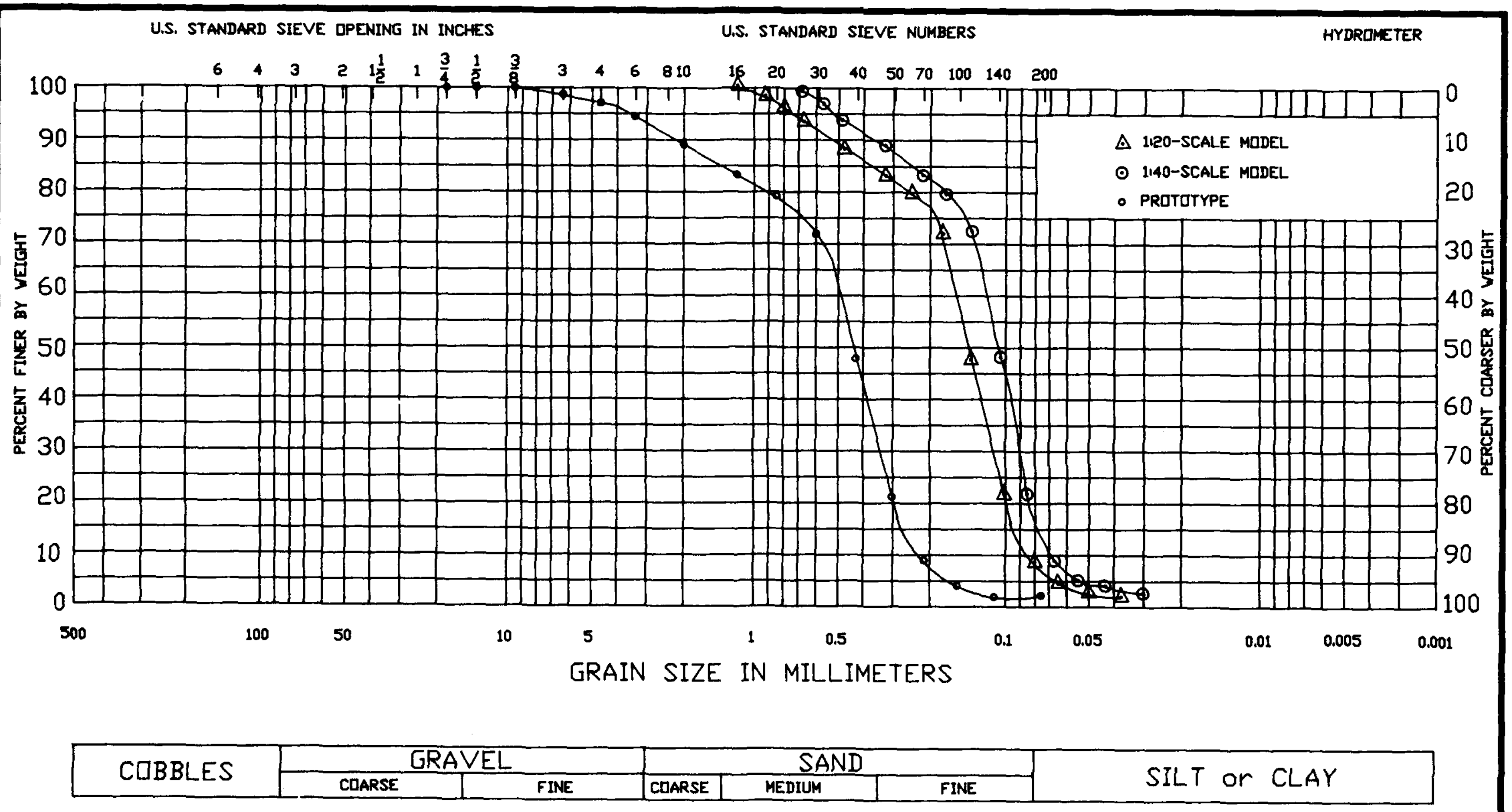


SETTLEMENT VELOCITY VERSUS PARTICLE DIAMETER UPSTREAM GRAVEL MATERIAL



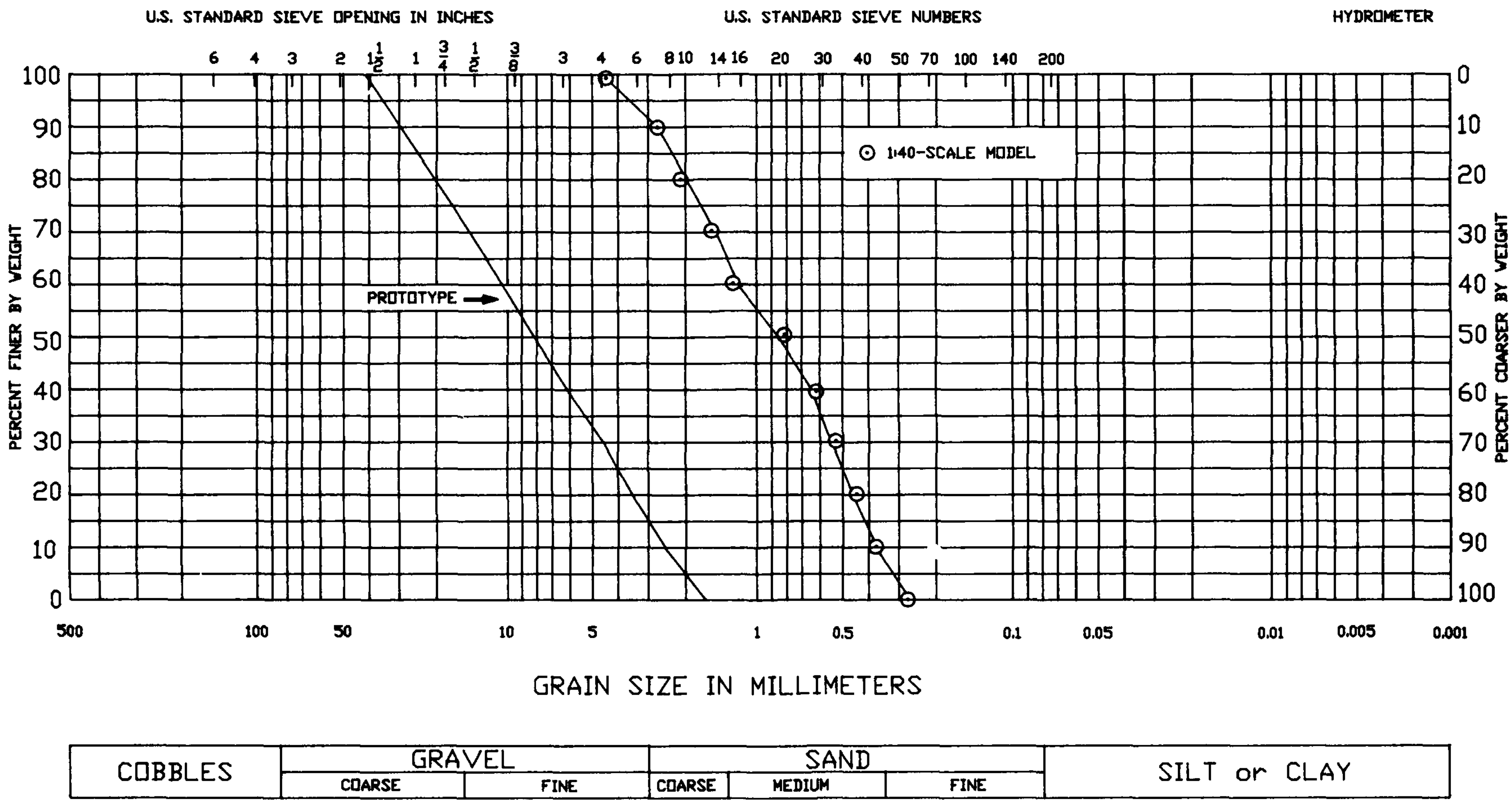
REYNOLDS NUMBER VERSUS
DRAG COEFFICIENT FOR
SPHERICAL PARTICLES

PLATE 11



GRAIN-SIZE DISTRIBUTION OF SAND FILL 1:20- AND 1:40-SCALE MODELS

PLATE 12



GRAIN-SIZE DISTRIBUTION OF UPSTREAM GRAVEL
1:40-SCALE MODEL

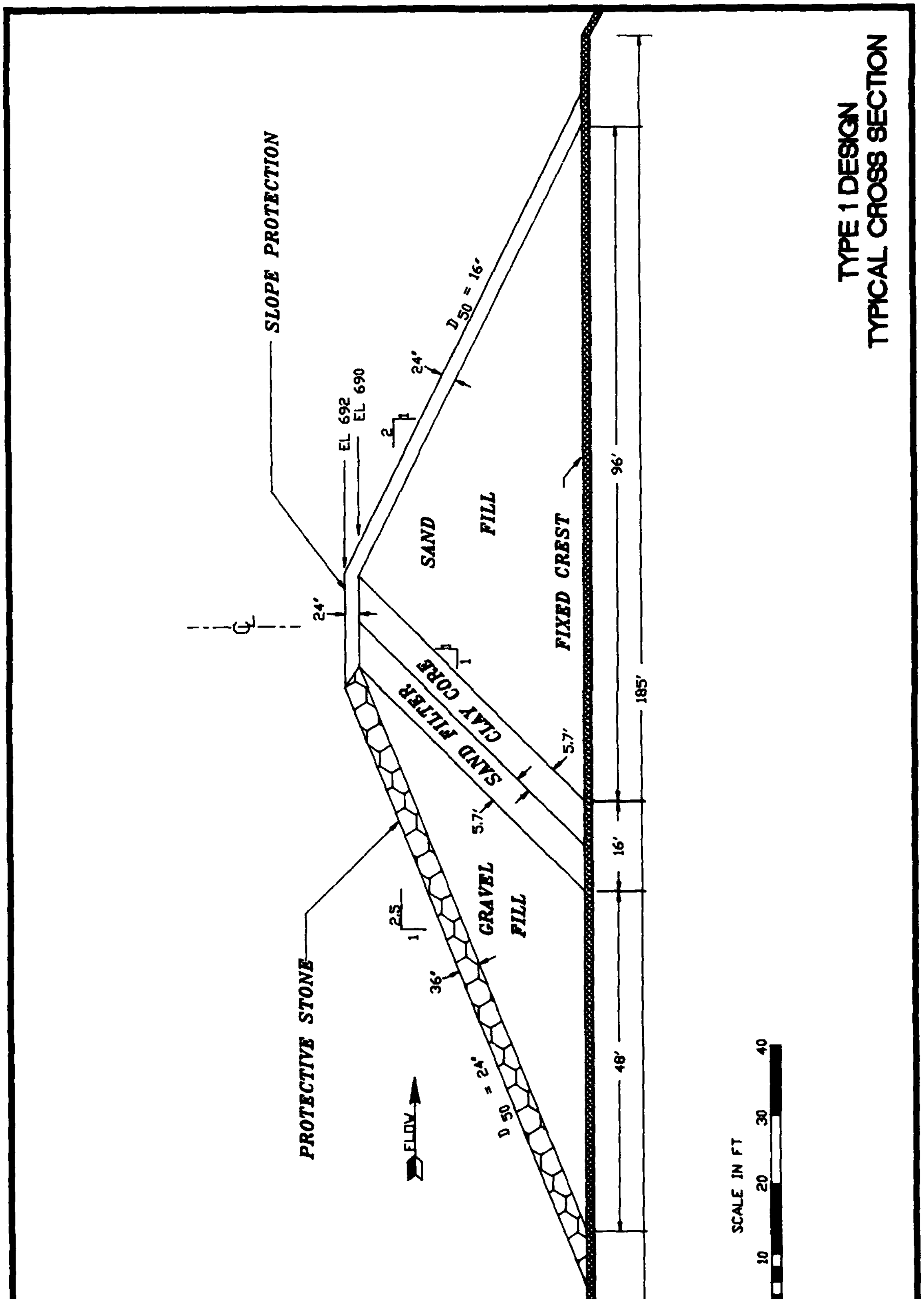
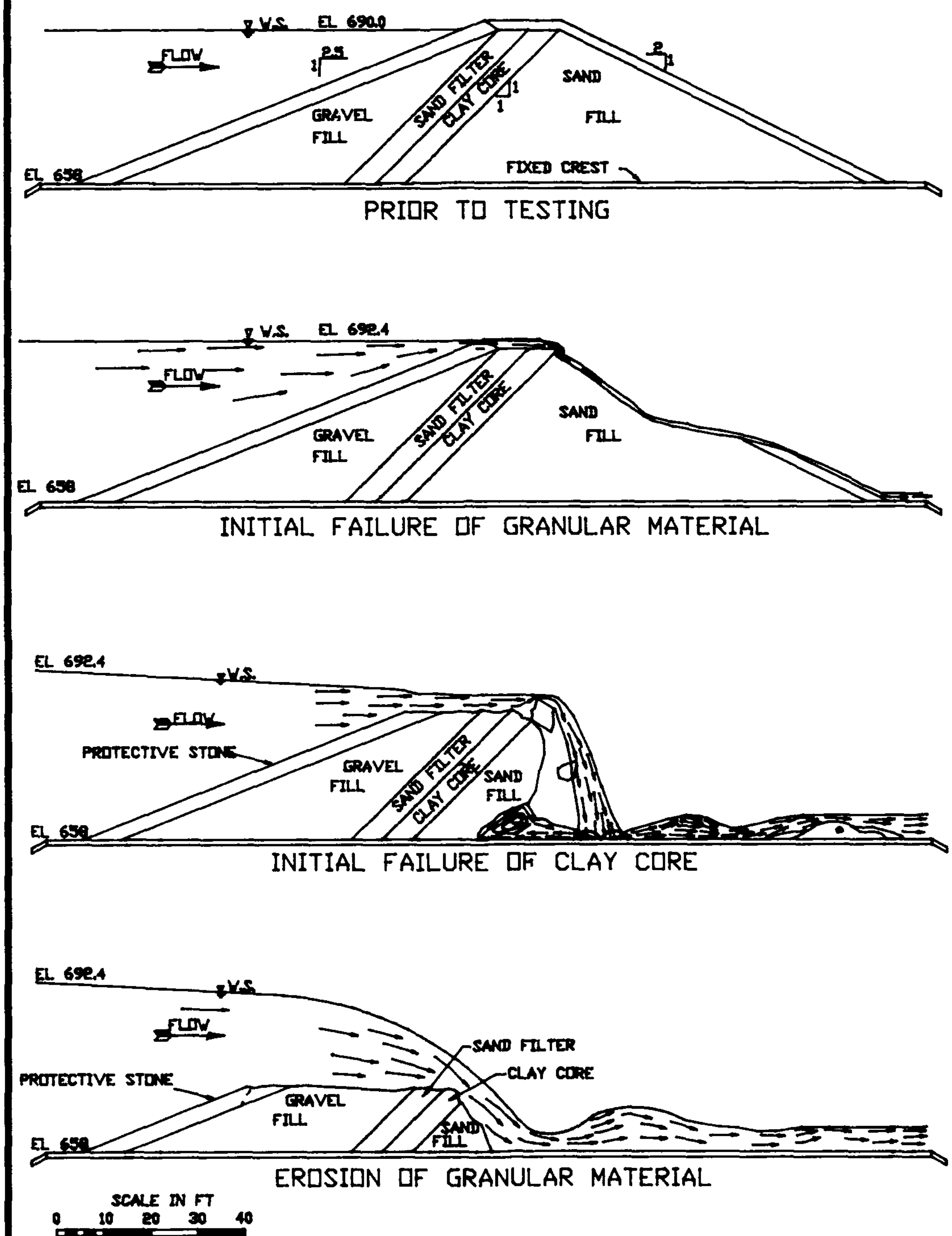
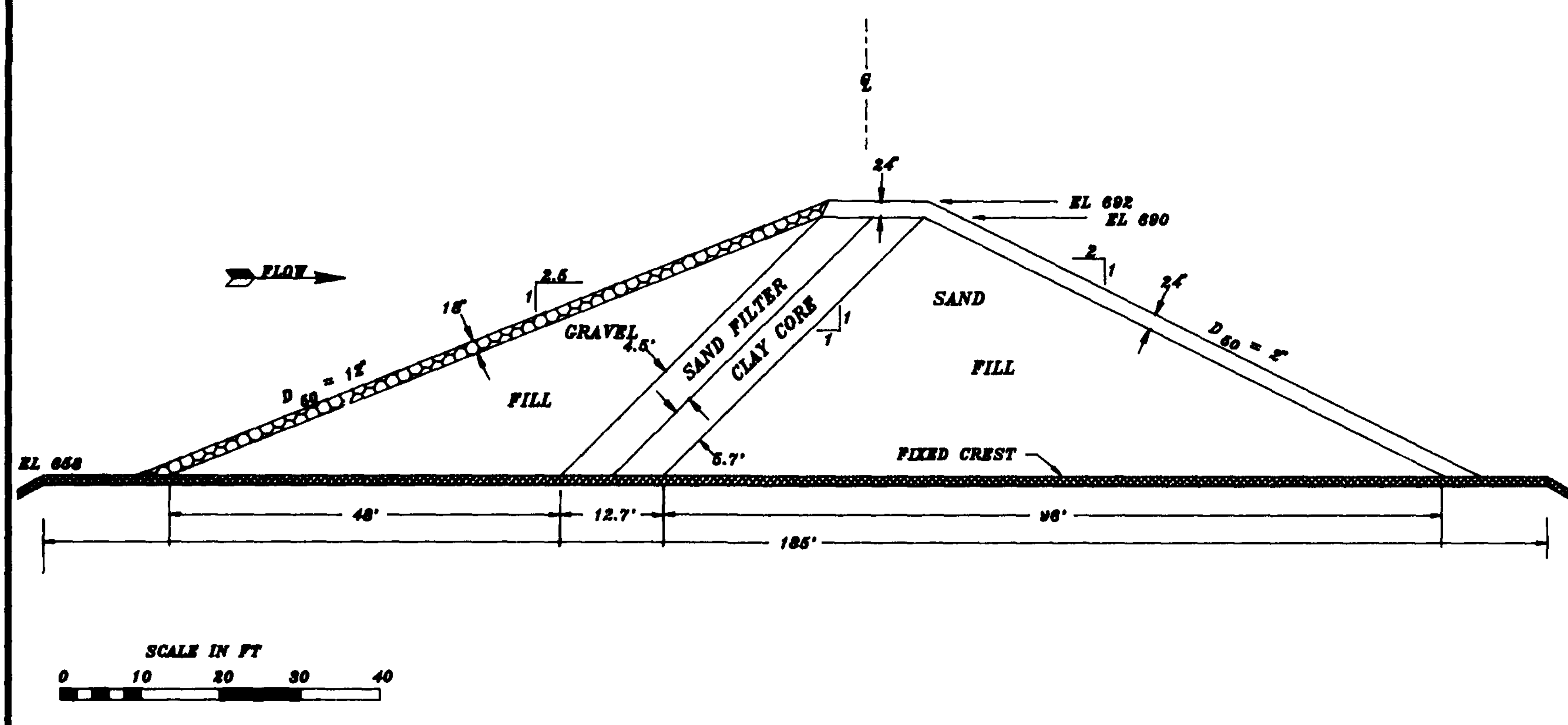

TYPE 1 DESIGN
TYPICAL CROSS SECTION
SLOPE PROTECTION
EL 692
EL 690
24'
D_{50} = 16'
2
1
SAND
FILL
FIXED CREST
CLAY CORE
SAND FILTER
5.7'
5.7'
PROTECTIVE STONE
GRAVEL
FILL
2.5
1
36'
D 50 = 24'
FLOW
EL 658
48'
16'
185'
96'
SCALE IN FT
0
10
20
30
40

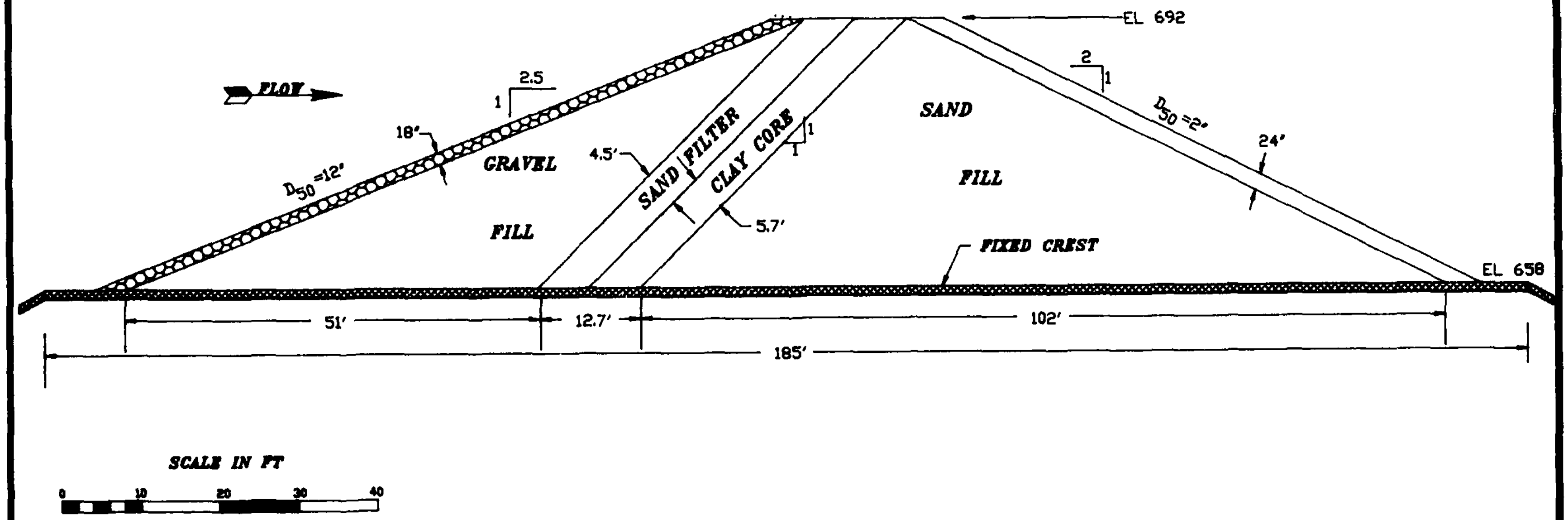
PLATE 14



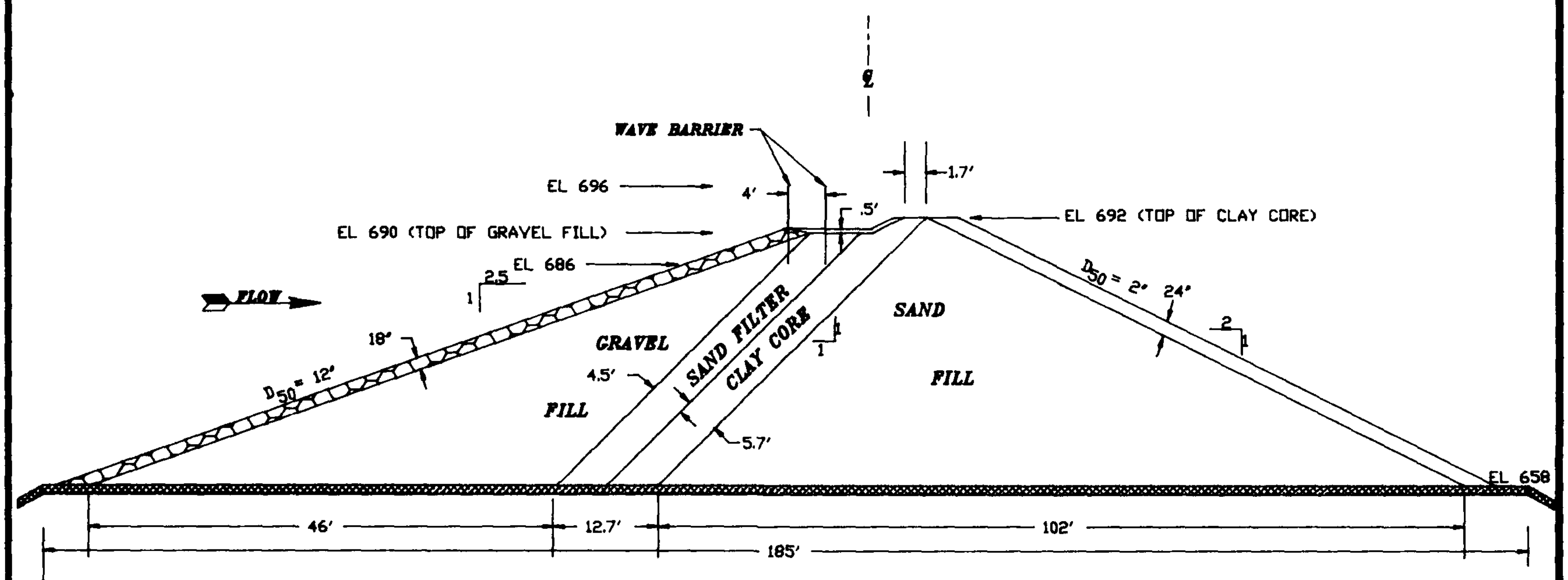
TYPICAL FAILURE SEQUENCE



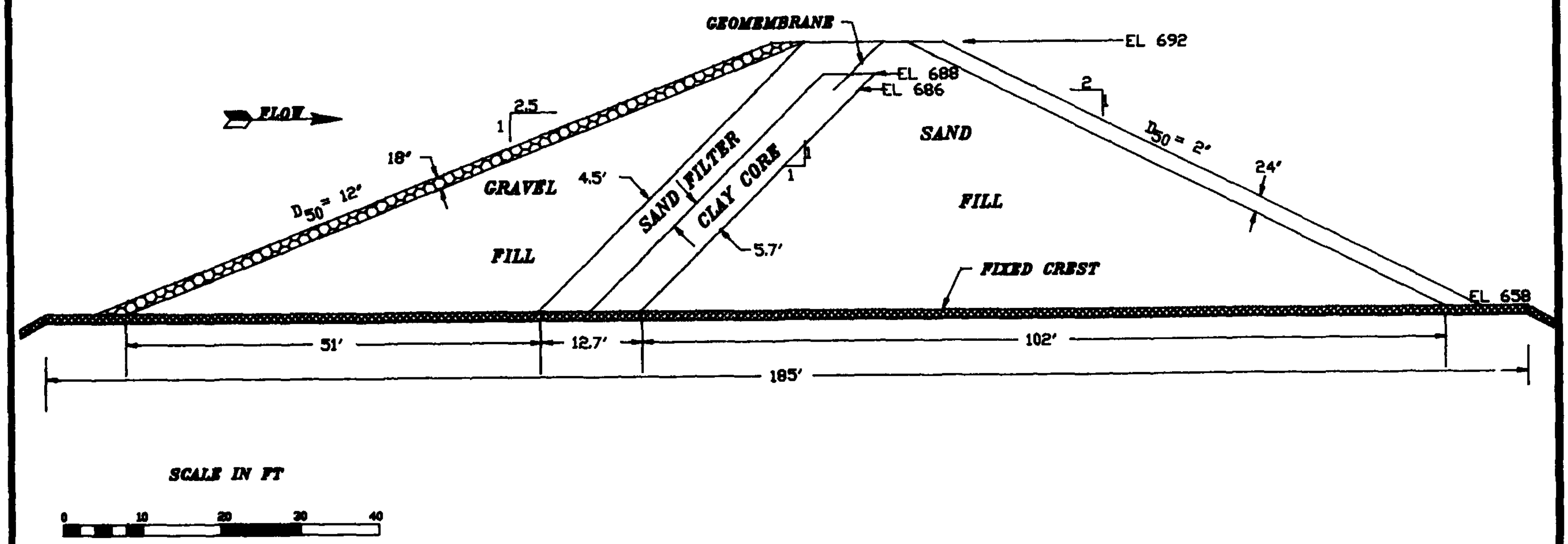
TYPE 2 DESIGN
TYPICAL CROSS SECTION



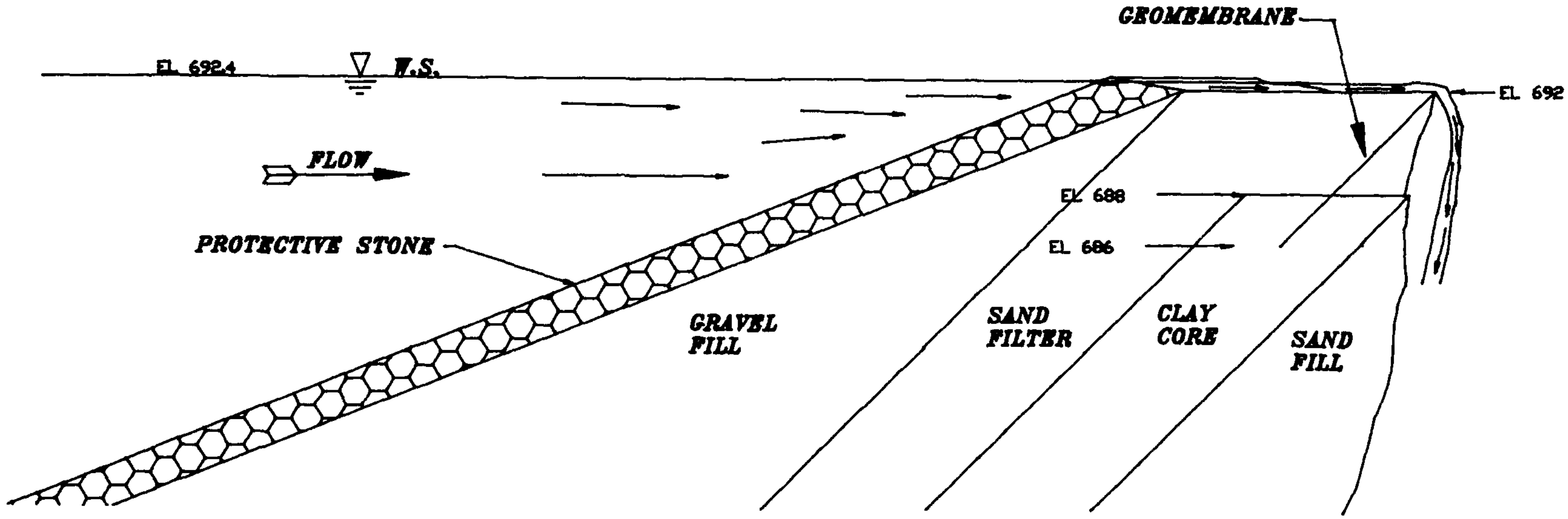
TYPE 3 DESIGN
TYPICAL CROSS SECTION



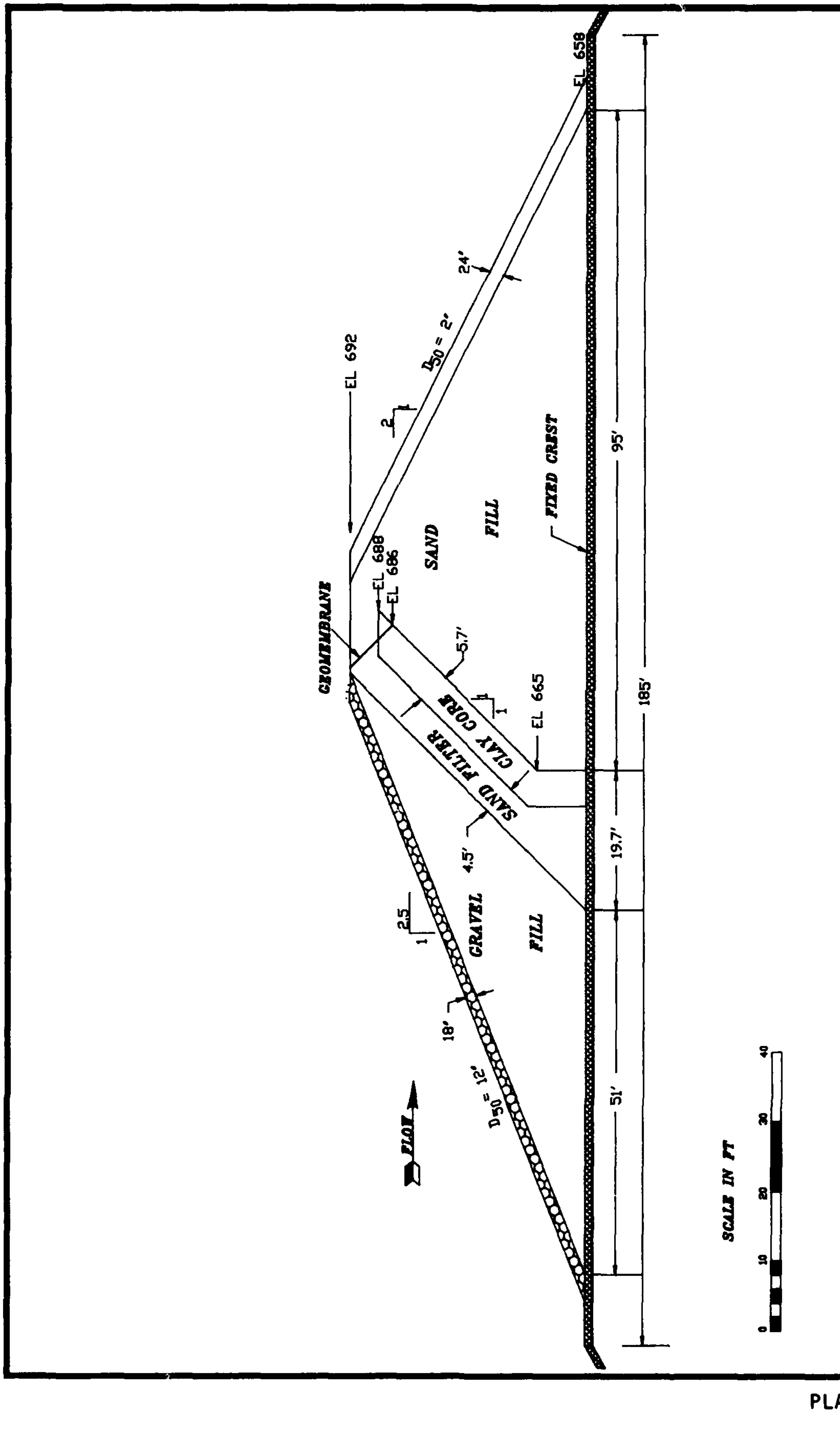
TYPE 4 DESIGN
TYPICAL CROSS SECTION



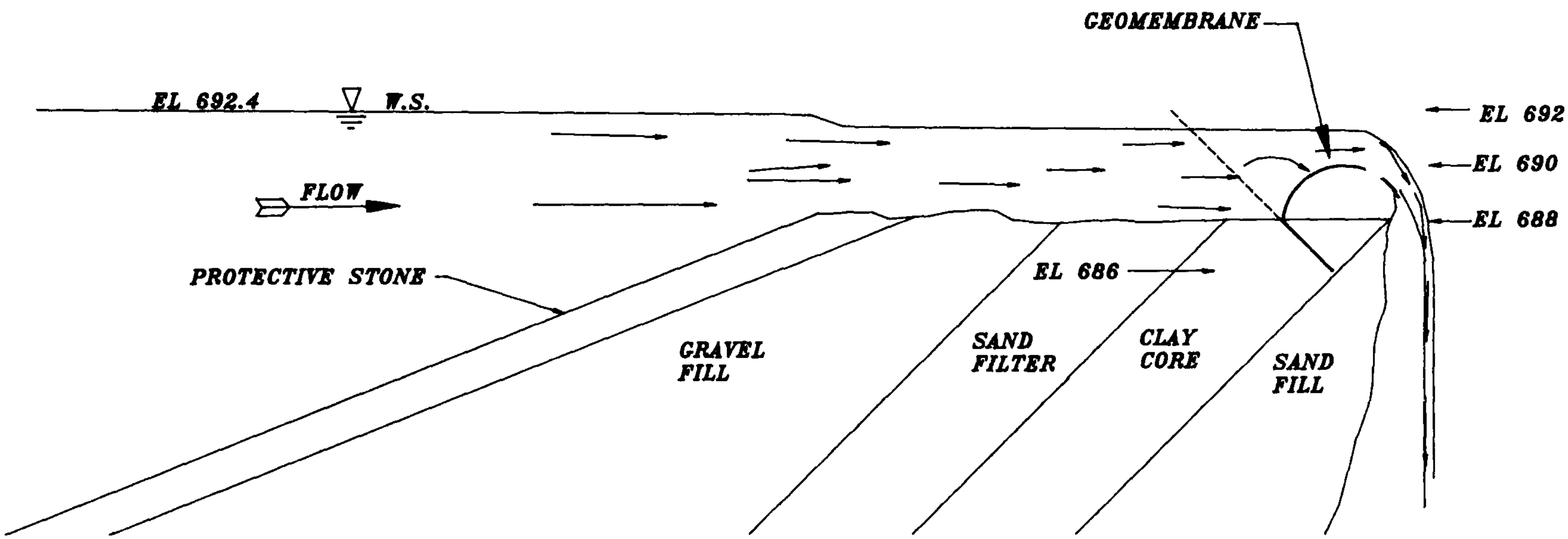
TYPE 5 DESIGN
TYPICAL CROSS SECTION



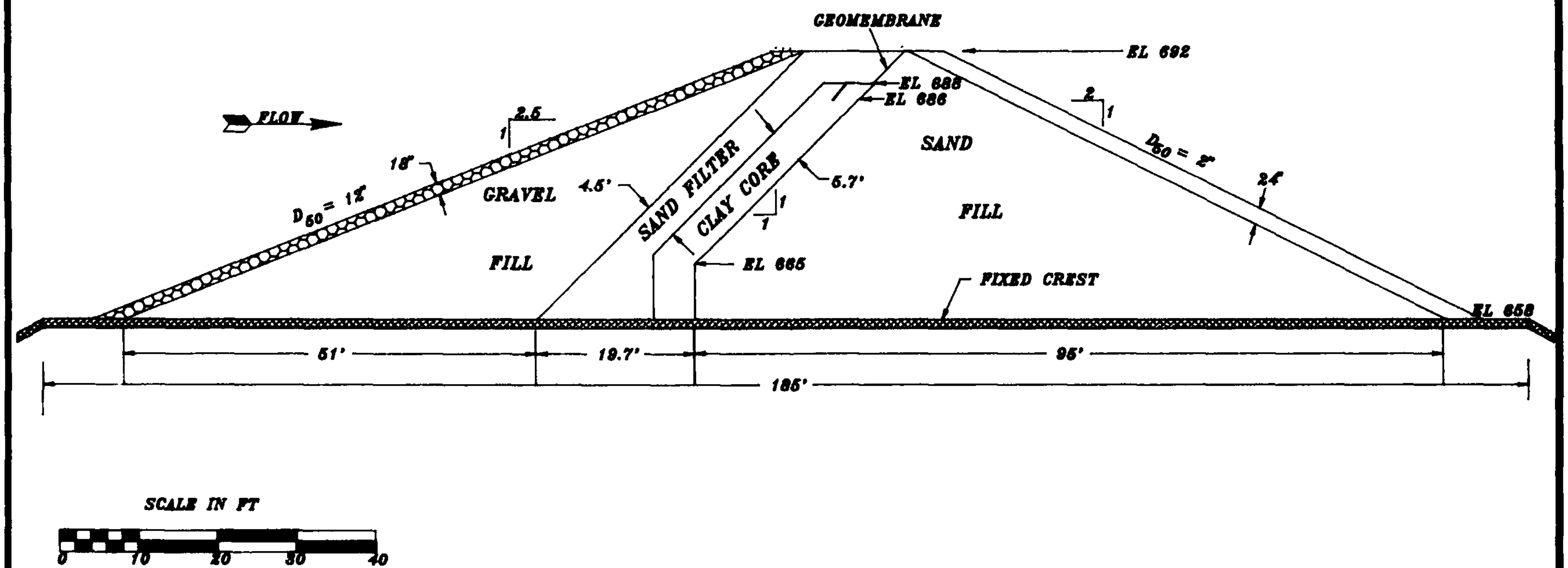
TYPE 5 DESIGN
STABLE IMPERVIOUS GEOMEMBRANE



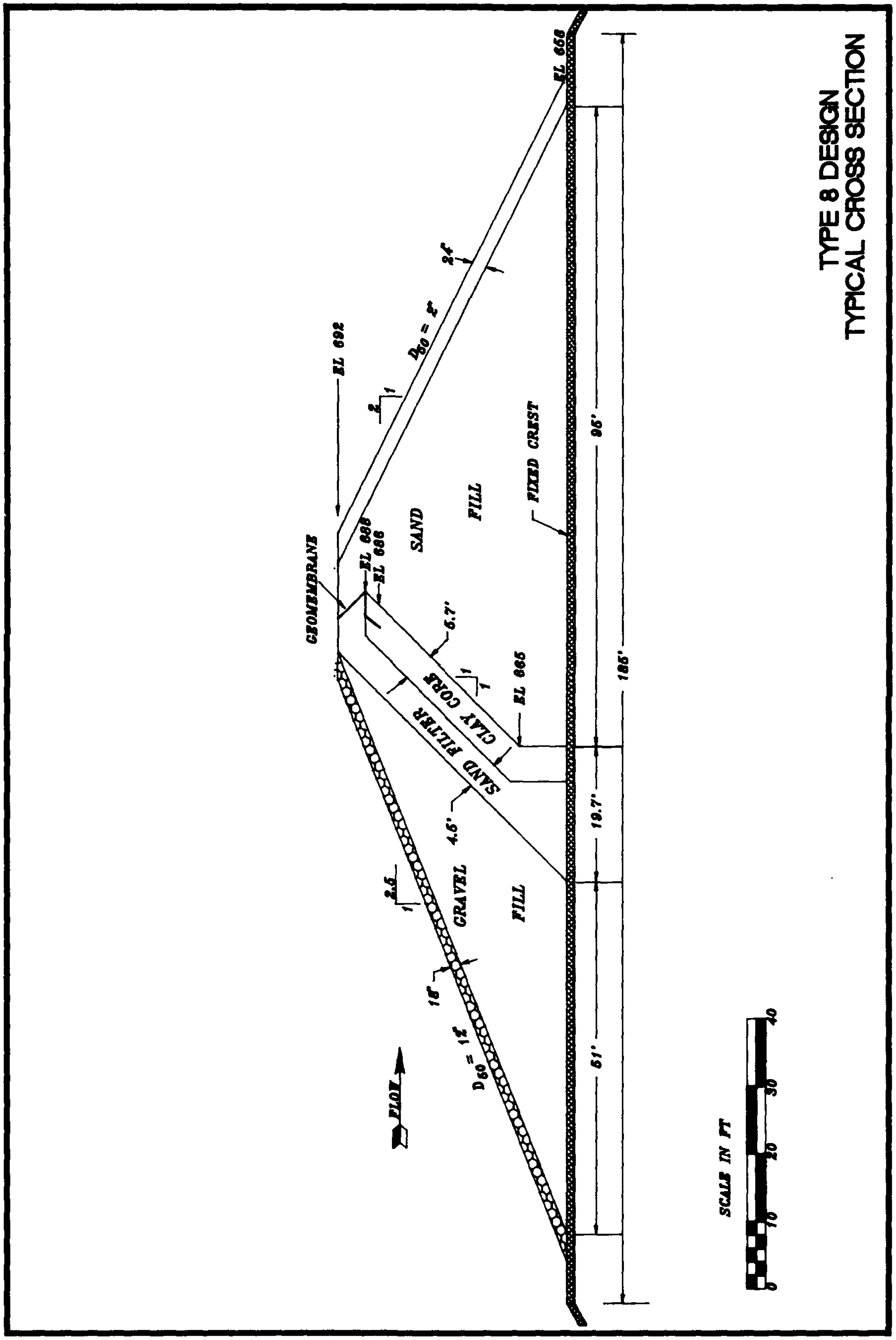
TYPE 6 DESIGN
TYPICAL CROSS SECTION



TYPE 6 DESIGN
PARTIAL FAILURE OF IMPERVIOUS GEOMEMBRANE

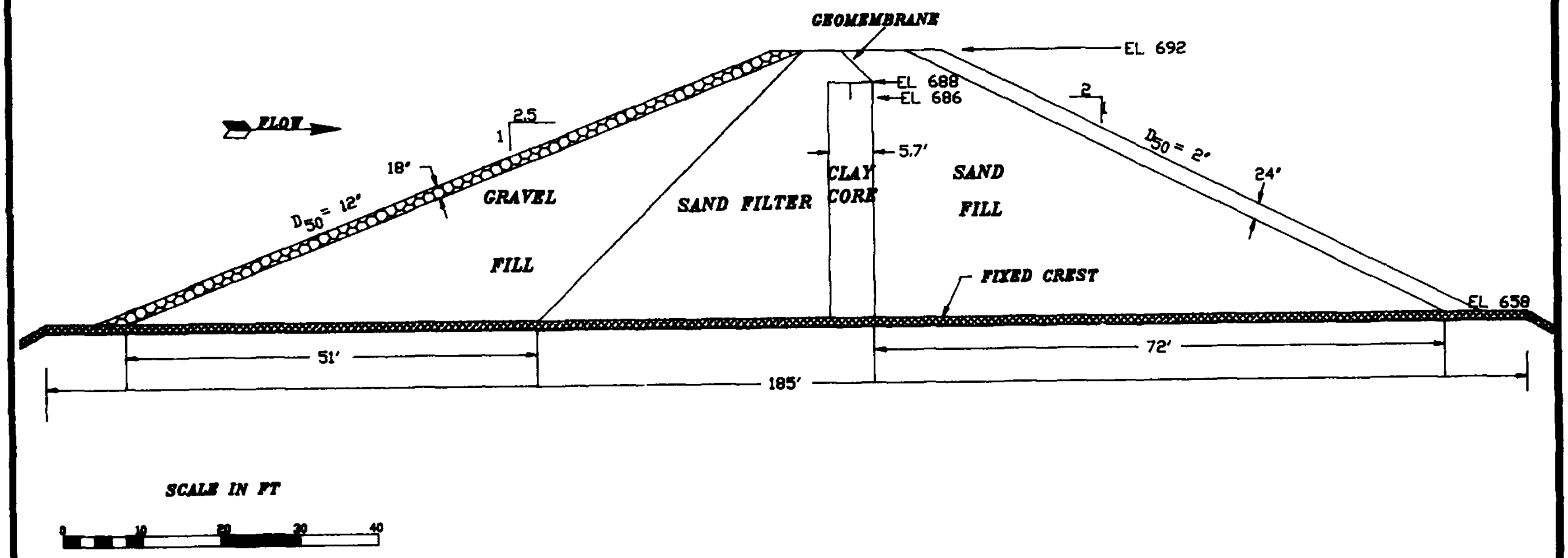


TYPE 7 DESIGN
TYPICAL CROSS SECTION



TYPE 8 DESIGN
TYPICAL CROSS SECTION
EL 658
EL 692
EL 688
EL 686
EL 665
GEOMEMBRANE
SAND
FILL
GRAVEL
FILL
CLAY CORE
SAND FILTER
FIXED CREST
D_{50} = 2'
D_{50} = 1'
24"
18"
5.7'
4.6'
2
1
2.5
1
1
1
95'
185'
19.7'
51'
FLOW
SCALE IN FT
0
10
20
30
40

PLATE 24



TYPE 9 DESIGN
TYPICAL CROSS SECTION

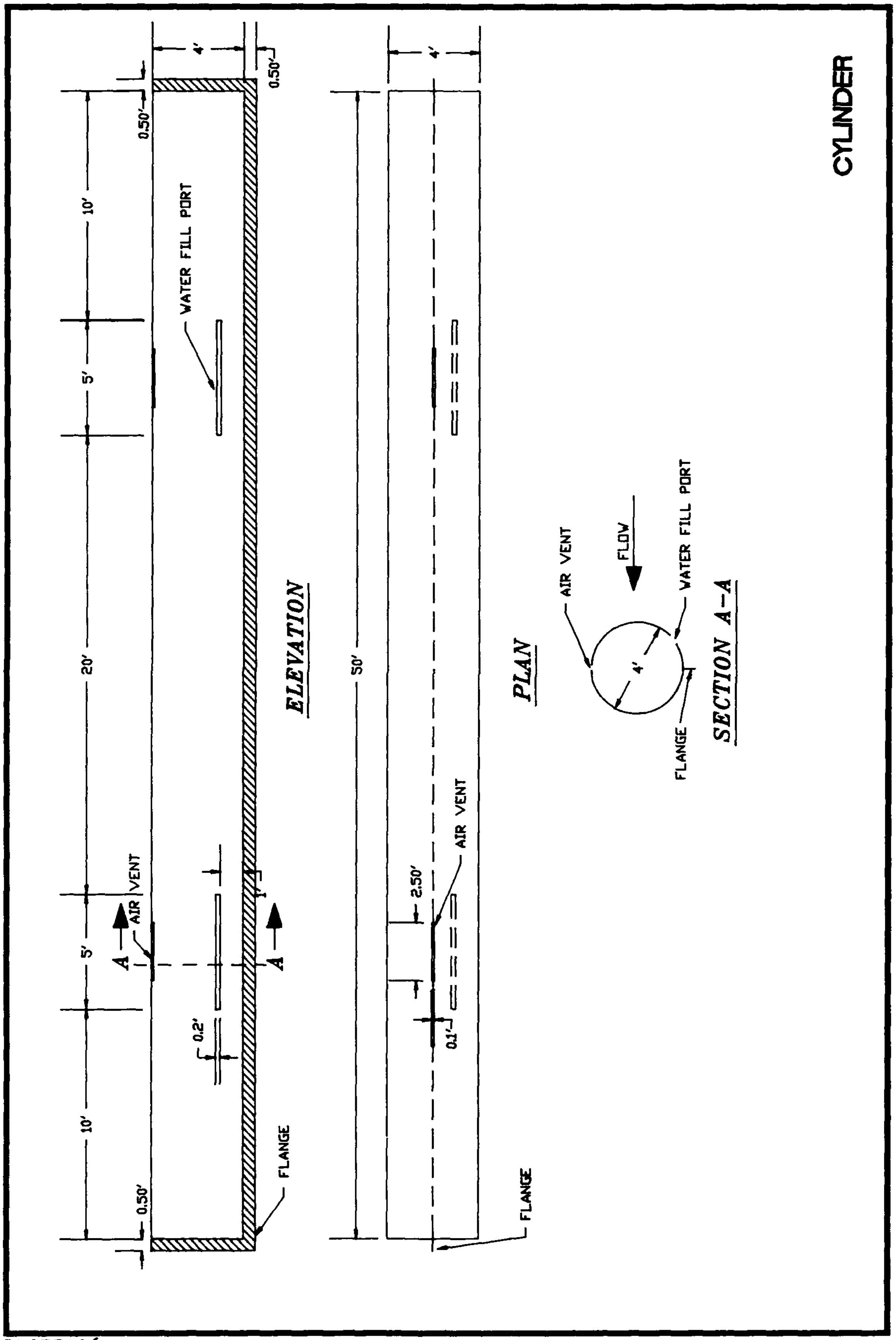
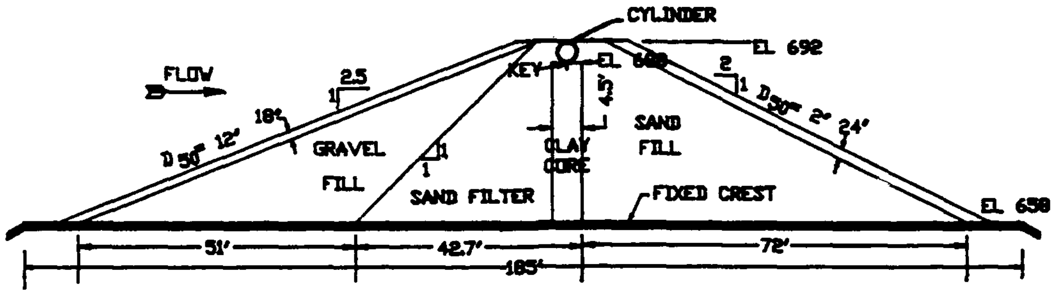
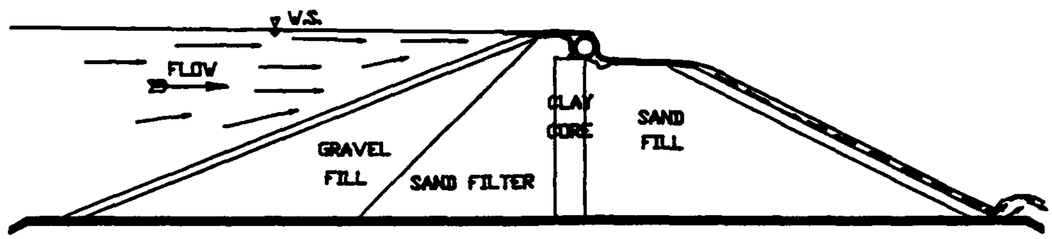

CYLINDER
ELEVATION
PLAN
SECTION A-A
WATER FILL PORT
AIR VENT
FLANGE
FLOW
A
10'
5'
20'
50'
4'
0.50'
0.2'
0.1'
2.50'
1'

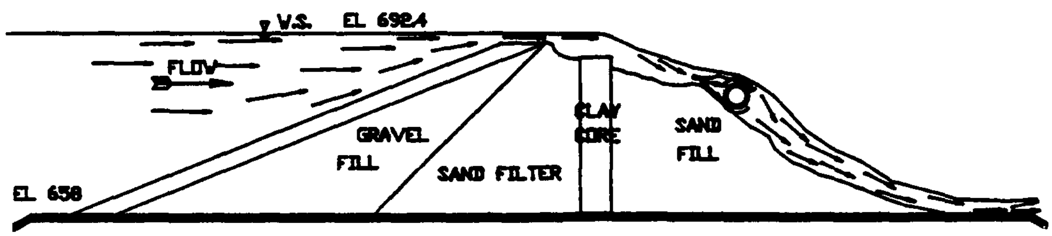
PLATE 26



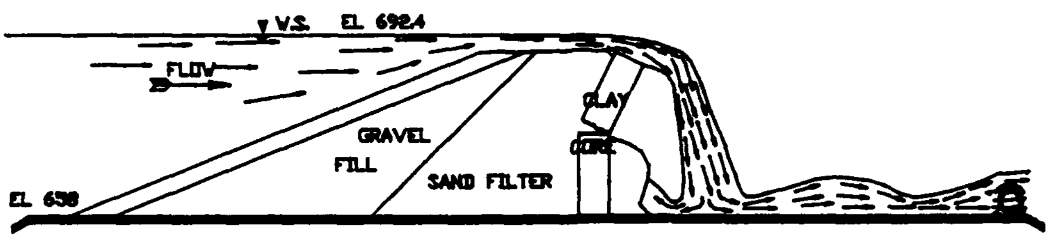
TYPICAL CROSS SECTION



INITIAL FAILURE

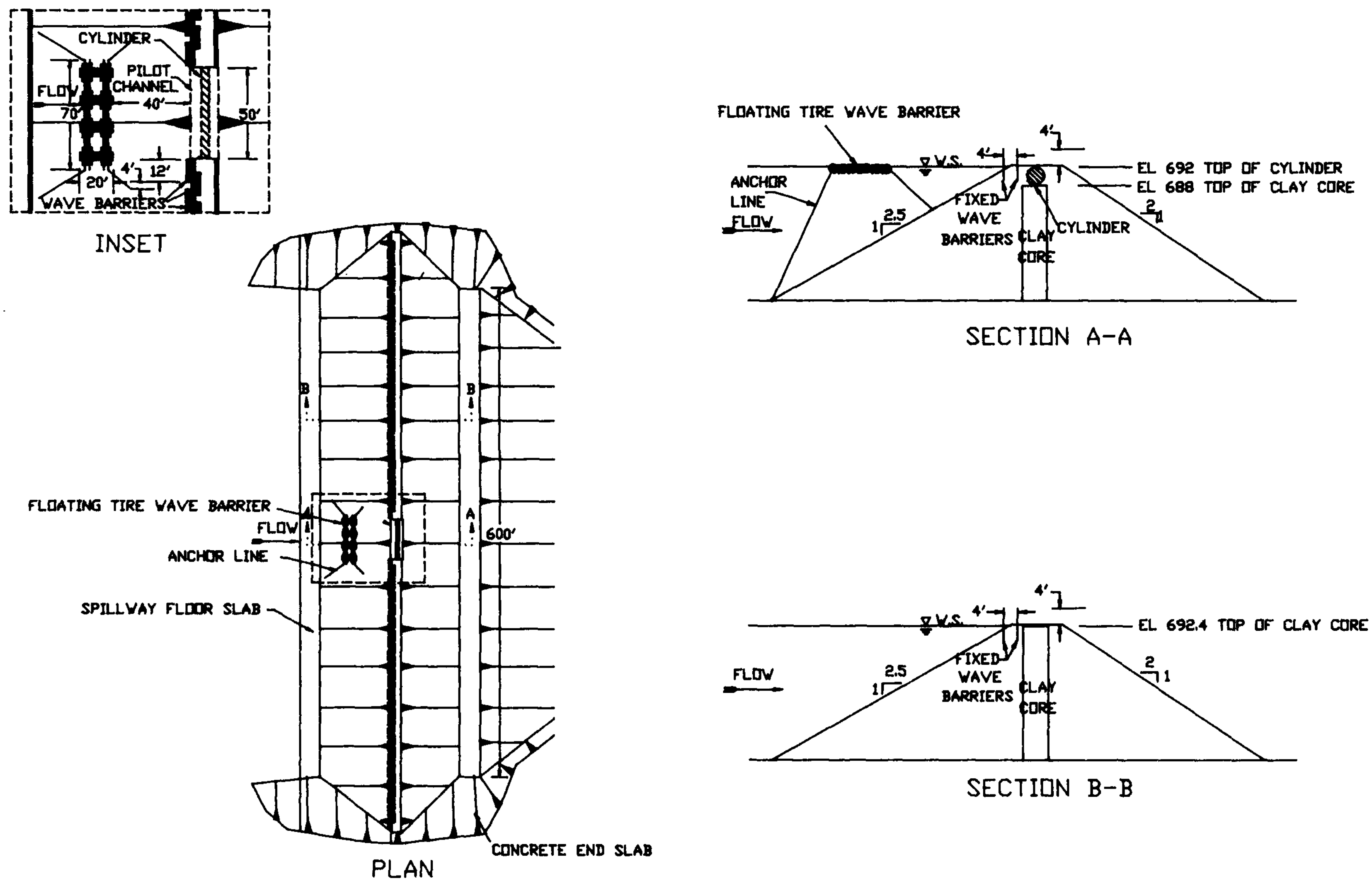


PROGRESSIVE FAILURE OF GRANULAR MATERIAL

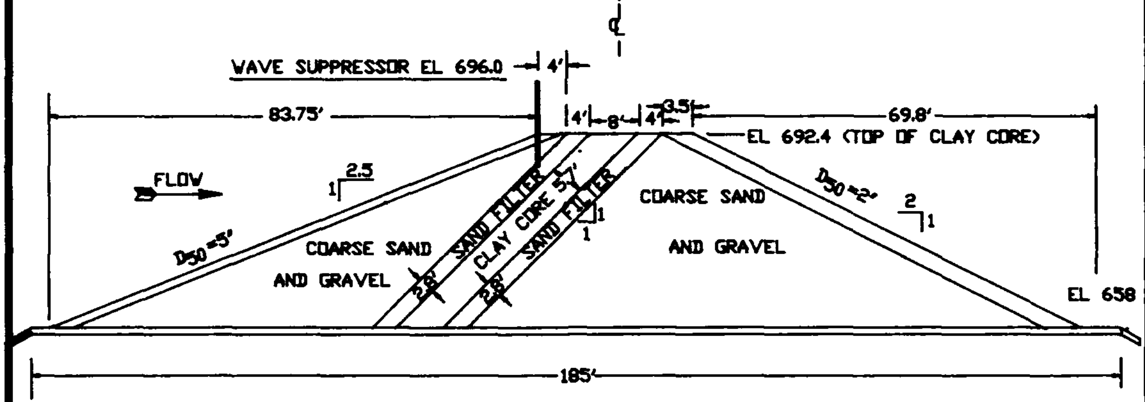


FAILURE OF CLAY CORE

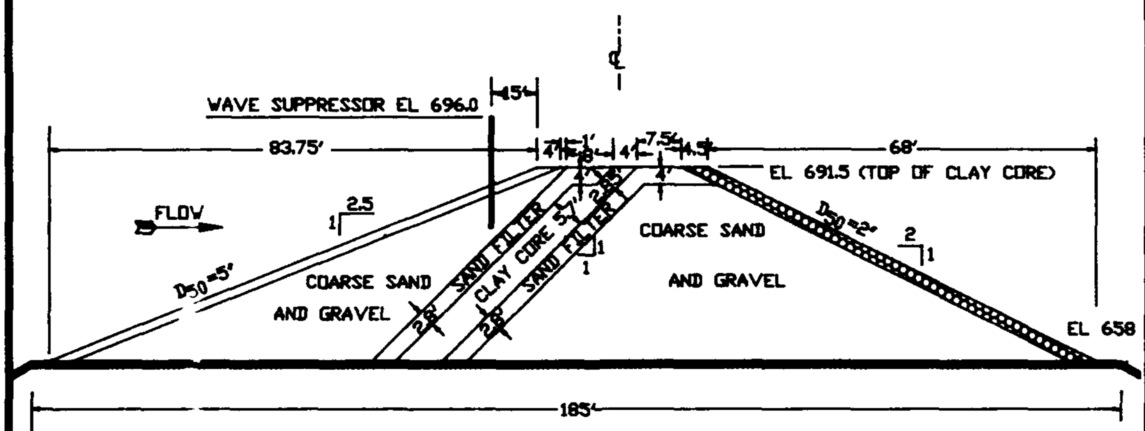
TYPE 10 DESIGN
TYPICAL CYLINDER FAILURE



TYPE 11 DESIGN

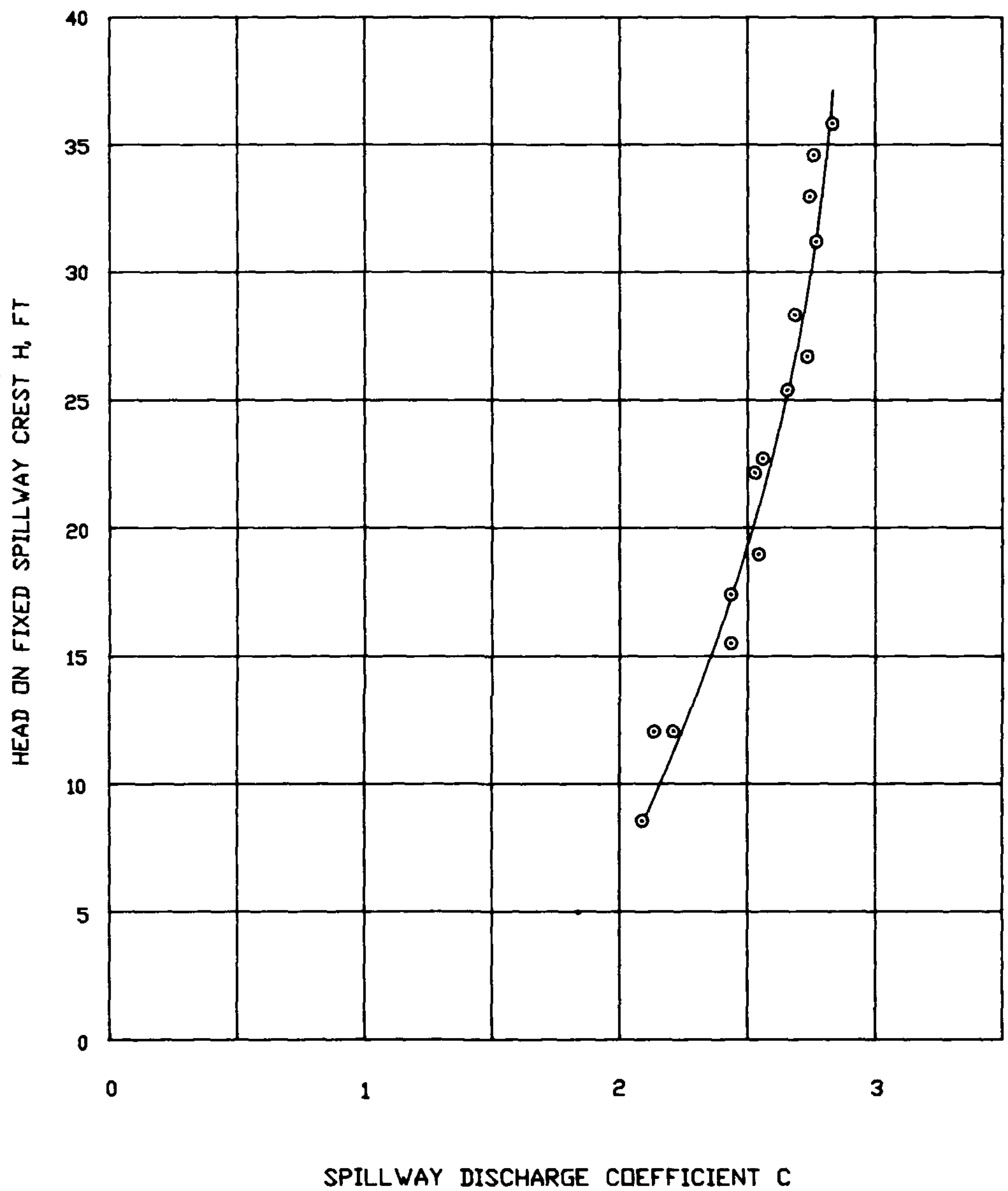


EMBANKMENT

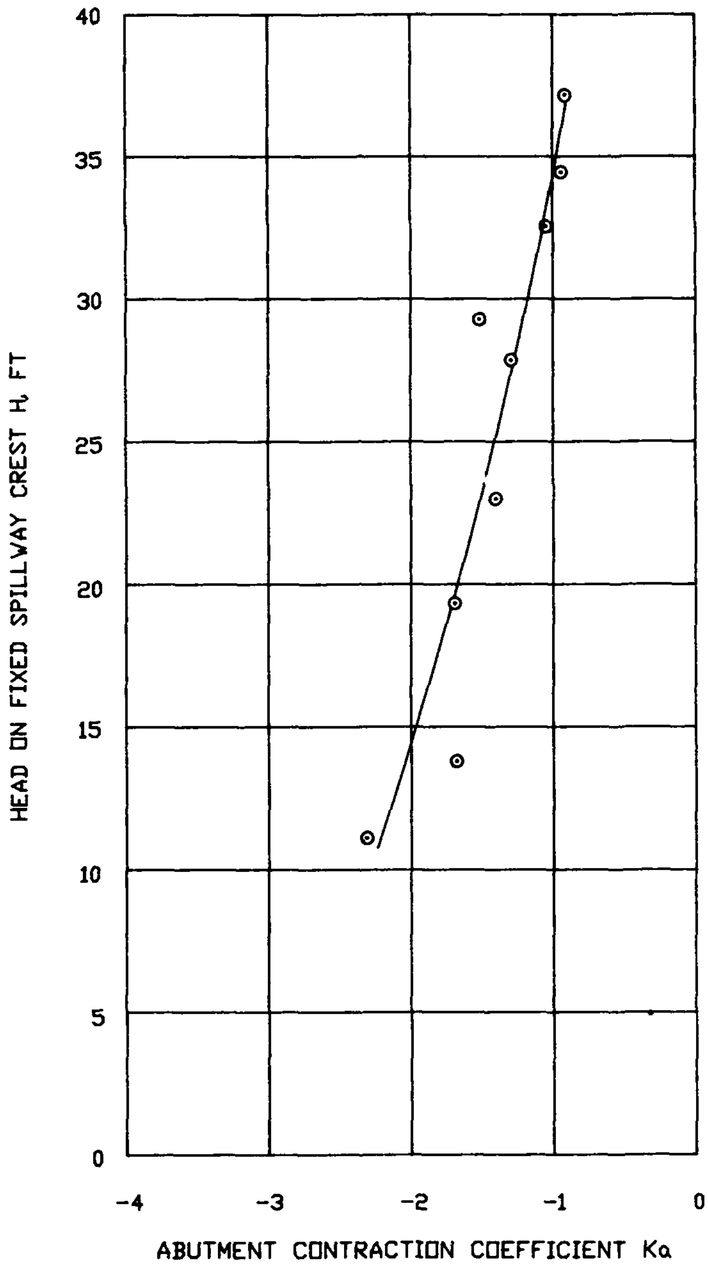


PILOT CHANNEL

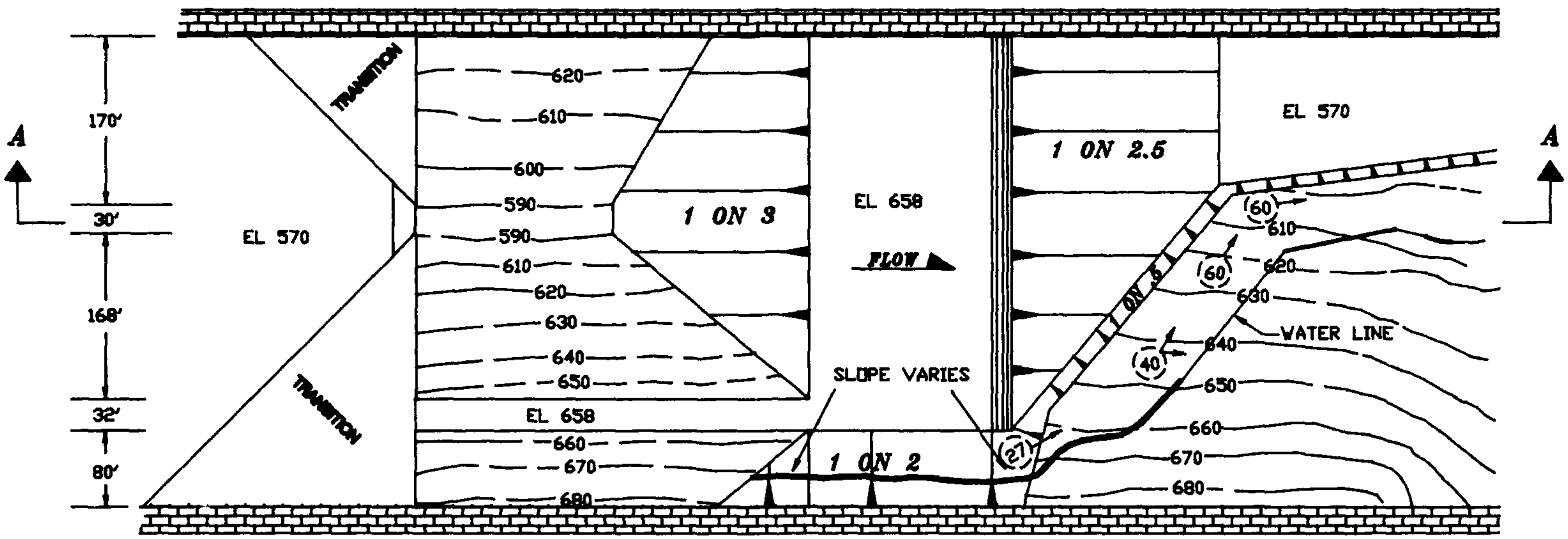
TYPE 12 DESIGN
TYPICAL CROSS SECTIONS



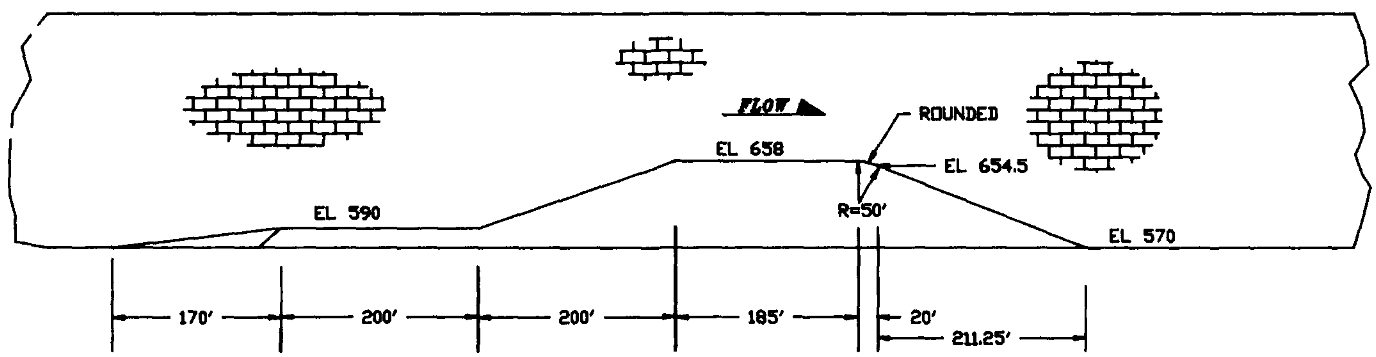
FIXED SPILLWAY DISCHARGE COEFFICIENT C



FIXED SPILLWAY ABUTMENT CONTRACTION COEFFICIENT Ka



PLAN



NOTE: VELOCITIES (60) ARE IN FEET PER SECOND MEASURED 2 FT ABOVE THE BOTTOM

SECTION A-A

FIXED CREST VELOCITIES
DISCHARGE 393,000 CFS

Waterways Experiment Station Cataloging-in-Publication Data

Fletcher, Bobby P.

Center Hill fuseplug spillway, Caney Fork River, Tennessee : hydraulic model investigation / by Bobby P. Fletcher and Paul A. Gilbert ; prepared for US Army Engineer District, Nashville.

77 p. : ill. ; 28 cm. — (Technical report ; HL-92-15)

Includes bibliographical references.

1. Spillways — Design and construction — Models. 2. Hydraulic structures — Tennessee — Caney Fork River. 3. Center Hill Dam (Tenn.) 4. Hydraulic models. I. Gilbert, Paul A. II. United States. Army. Corps of Engineers. Nashville District. III. U.S. Army Engineer Waterways Experiment Station. IV. Title. V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-92-15.

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